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Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

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Journal of the
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EXPERIENCES WITH LOESS AS FOUNDATION MATERIAL

William A. Clevenger,[1] A.M. ASCE

SYNOPSIS

Certain characteristic properties governing the behavior of loess as a foundation material have been defined through extensive laboratory and field studies conducted by the Bureau of Reclamation. These studies were primarily limited to the loess or loess-like materials occurring in the Missouri River Basin of central western United States. In this paper, broad generalizations of many pertinent properties of loess are presented and their significance is pointed out by discussions of specific typical experiences with loess as foundation material. Some interesting data gathered by the writer on residence foundation failures in Colorado are described, as well as results of laboratory and field studies of the properties of the loess connected with these failures.

INTRODUCTION

True loess is a very loose, wind-deposited soil covering vast areas of several continents, including North America. It is generally composed of uniform, silt-sized particles which were loosely deposited, and are bonded together with relatively small fractions of clay forming the typical loess "structure." Normally, loess has high shearing resistance and will withstand high loadings without great settlement when natural moisture contents are low. However, upon wetting, the clay bond tends to soften and cause collapse of the loess structure inducing large settlement under low loading, and loss of shearing strength.

A thorough basic knowledge of the engineering properties of loess is necessary to safely utilize this material as foundation. Much has been written on the origin and deposition of loess, its mineralogical composition, and its engineering properties as defined by laboratory and field tests, but there

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is little published on the results of actual experiences with loess as foundation material. The primary purpose of this paper is to present a few typical examples of such experiences and to correlate them with the properties of the materials involved, as determined by laboratory test. While the experiences presented are quite varied, it is felt that such information may be of benefit to those who are confronted with the foundation problems associated with construction in loess areas.

Much of the laboratory test data contained in this paper was obtained by the Bureau of Reclamation in connection with construction of hydraulic works in the Missouri River Basin, areas of Nebraska and Kansas, and have been reported in detail by Holtz and Gibbs.[2] Data applying to loess deposits in the Denver, Colorado area were furnished by resident soil consultants. Comparisons are made throughout this paper of the properties of the particular loess being discussed to those of the average Missouri River Basin loess, in order to bring out peculiarities of individual deposits.

Index Properties of Loess—Missouri River Basin Area

Appearance and Structure

The typical undisturbed Missouri Basin Loess is a tan to light brown, crumbly, lightweight material containing many visible voids and root holes, as shown in the photograph (Figure 1a). At low natural moisture, it is fairly brittle but can usually be easily powdered between the fingers. Few individual particles can be seen with the unaided eye, and practically no stratification is visible. Most of the rootlike channels trend in a vertical direction, thereby giving vertical cleavage to massive loessial deposits. This can be observed in the field in the splitting off of columnar sections of dried loess in steep-sided gullies.

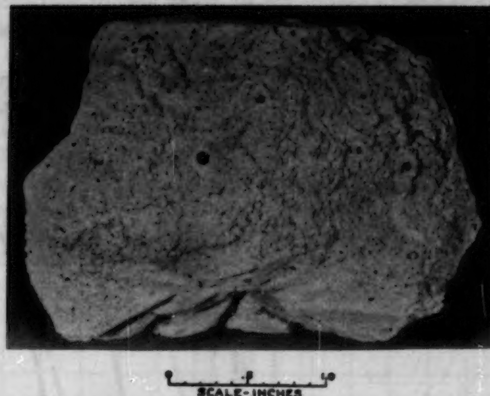
Gradation and Plasticity

Loess typically contains mostly silt-sized particles of quartz and feldspars bonded together by a relatively small proportion of montmorillonite-type clay. Most of the grains are between 0.019 and 0.074 mm except in the more clayey loesses, as indicated in Figure 1b. About three-fourths of the samples investigated from the Missouri Basin area fell within the boundaries indicated by "silty loess," one-fifth were classified "clayey loess," and about one-twentieth was "sandy loess." Practically all of the loess has some plasticity when remolded, as indicated by the Atterberg limits chart (Figure 1b). The relatively small amount of clay present has a significant influence on the engineering properties of the loess.

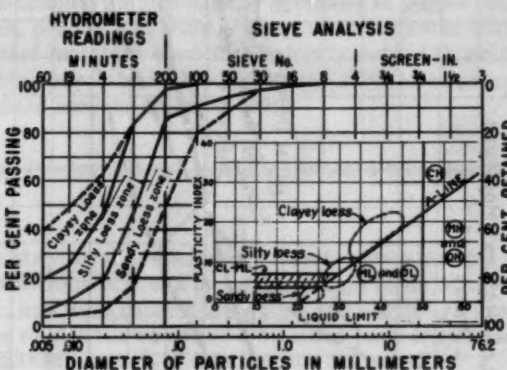
Undisturbed Density

In the natural state, Missouri Basin loess is normally very loose because of its method of deposition, and because it has not been subjected to sufficient saturation in nature to produce "breakdown" of the typical structure. Undisturbed densities of true, wind-deposited loess normally range from about 75

2. "Consolidation and Related Properties of Loessial Soils," by W. G. Holtz and H. J. Gibbs, Special Publication No. 126, ASTM, 1952.



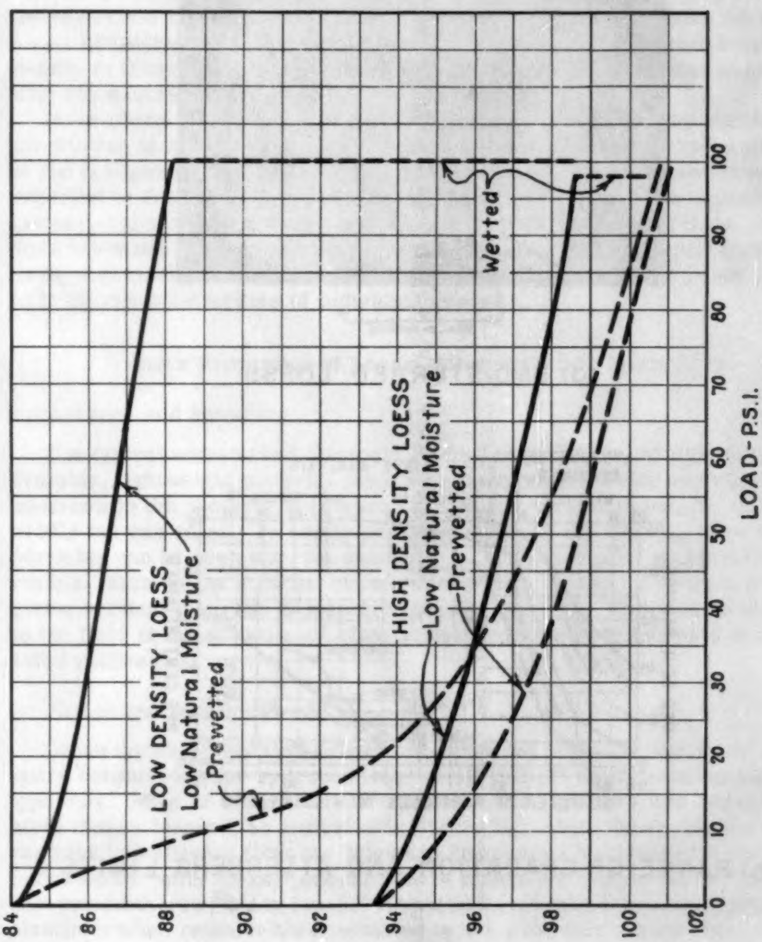
(a) UNDISTURBED LOESS



(b) RANGE OF GRADATION AND ATTERBERG LIMITS

APPEARANCE AND IDENTIFICATION MISSOURI RIVER BASIN LOESS

Figure 1



TYPICAL CONSOLIDATION CURVES
MISSOURI RIVER BASIN LOESS

Figure 2

to 85 pounds per cubic foot in this area. If the material has been wetted and consolidated, or has been reworked, the natural density is higher, sometimes as much as 100 pcf or more. In unusual cases, the wind-deposited material has been found as low as 65 pcf in density. The Bureau's studies have shown that the natural density is perhaps the most important index property of loess, in that the ultimate settlement which can be expected and the shearing resistance of the material after wetting are dependent largely on the natural density. Large settlements and low shear resistance can be anticipated for Missouri Basin loess at density of 80 pcf or less, while the soil above 90 pcf will settle a relatively small amount, and will have fairly high shearing strength. Between 80 and 90 pcf the properties are transitional. Standard Proctor maximum densities vary from about 105 to 110 pcf for the loess to which the above criteria apply.

Natural Moisture

Moisture contents of the undisturbed loess are usually in the order of 10 percent and, regardless of the density, the supporting capacity of loess at this moisture is high. With increasing natural moisture to about 15 percent, the supporting capacity is reduced only a small amount, but further increase in moisture is associated with pronounced decrease in supporting capacity. Because of the open, porous structure, Missouri Basin loess will retain only about 25 to 28 percent moisture even if thoroughly wetted by artificial means. At moisture contents above about 15 percent, the structural properties of loess appear to depend largely on the density.

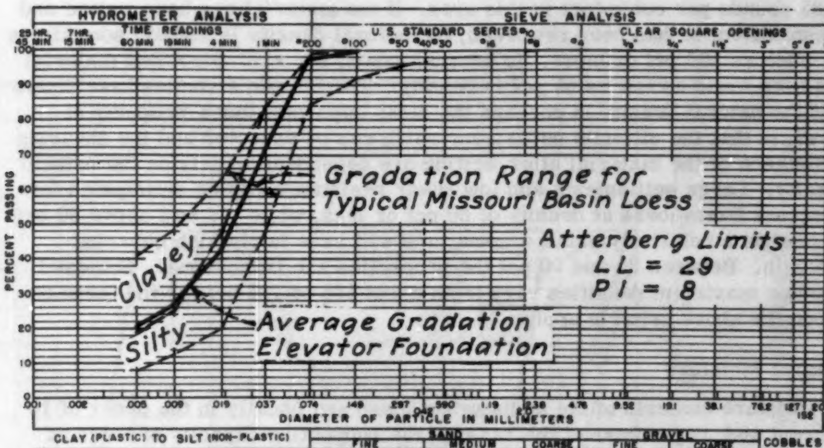
Structural Properties of Loess—Missouri River Basin Area

Consolidation

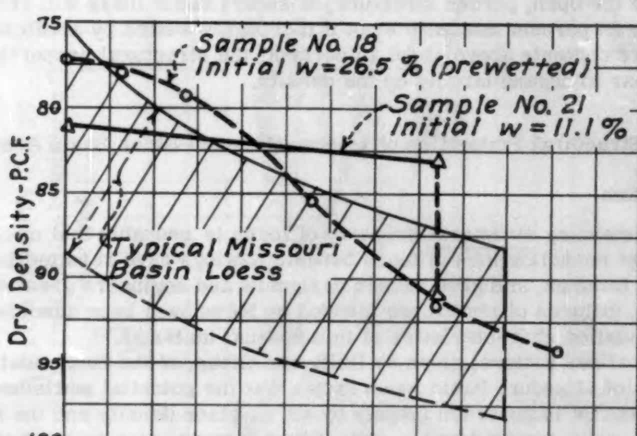
The outstanding structural property of loess is probably that of consolidation. Large consolidations of loess beneath footings have resulted in numerous foundation failures, and have caused designers and engineers great concern. Invariably, failures of structures founded on loess have been associated with the consolidation characteristics of this unusual material.

A generalized picture, given by Holtz and Gibbs, of the consolidation properties of Missouri Basin loess shows that the potential settlement of a loess foundation is governed largely by the in-place density and the highest moisture content attained by the soil. Other factors, such as gradation, past and present natural loadings, also have an effect but by comparison these are of relatively minor importance.

Several typical laboratory consolidation curves for test specimens of loess have been plotted in Figure 2 as load versus density to demonstrate the effect of in-place density, and of wetting on consolidation properties. Test specimens at low natural moisture consolidate comparatively little, whether at high or low density. Prewetted, low-density specimens consolidate excessively (15-20 percent), and high-density specimens either at natural moisture or prewetted, consolidate but little. The "effect of saturation" can be observed from the additional consolidation which results from wetting the specimens while under a 100 psi load. The approximate effect of saturation at other loadings can be estimated from such curves as these by calculating the difference in density between the natural and prewetted specimens at any particular load.



GRADATION AND ATTERBERG LIMITS



CONSOLIDATION CURVES

ELEVATOR FOUNDATION LOESS (KANSAS)

Figure 3

There have been numerous large engineering structures of various types founded on loess in the Missouri Basin area, most of which have performed satisfactorily, but some of which have failed. One example of unsatisfactory performance is for a grain elevator in Kansas, data for which was furnished the writer by a soils consultant who analyzed the foundation conditions. This elevator was completed during July several years ago. After heavy rains occurred in August of that year, the structure settled quite badly on the north side, causing very noticeable tilting in that direction. It was believed by those concerned, that water had ponded on the north side because of poor surface drainage and had caused the settlement. An attempt was made to correct the tilting by wetting beneath the southern part of the foundation, and loading this part of the bulkling to reduce the differential settlement. (To the writer, this appears to be a rather dangerous procedure in that uncontrolled shearing failure could have been induced.) The treatment was partially successful and the structure was tilted part way back.

Recently, investigations were initiated at this site to study the properties of the foundation materials which by this time were fairly dry. Undisturbed samples were obtained and identified as loess. Laboratory tests were performed to define the behavior of the loess under loading and wetting, and thus explain why the grain elevator settled as it did. The results of these tests (Figure 3) showed that the loess was typical of Missouri Basin loess, not only from the grain size and plasticity viewpoint, but also from the standpoint of consolidation characteristics. Sample No. 21, at low natural moisture content, subjected to increasing load increments, settled from 81 to 83 pcf density (2.4 percent) under the approximate elevator foundation pressure. When wetted while under this load, the specimen settled an additional 8 or 9 percent. The consolidation curve for Sample No. 18 which was wetted before loading shows that settlement of the wetted loess, or loess at high natural moisture, would have occurred at considerably lower loadings than the elevator foundation pressure.

This grain elevator does not appear to have moved appreciably since precautions were taken to prevent uncontrolled foundation wetting.

Shear Strength

A generalized picture of shearing strength properties, as determined by laboratory tests, is shown in Figure 4. These curves were developed from results of a great number of tests on samples ranging from clayey to sandy loess, and covering a wide range of densities and moistures. It was found that the moisture and density at time of test controlled the shearing strength. Differences in sand and clay content appeared to exert comparatively minor influence, and envelopes were generally parallel indicating that the coefficient of internal friction varied but little for different loesses. This might be expected for such uniform-grained materials. The variation in the location of the envelopes indicates the effect of density and moisture on the total average shearing strength. Wetted, low-density test specimens had almost zero shearing strength until they had been consolidated so that the soil grains were brought into closer contact. Approximately 10 psi effective normal stress was required to accomplish this in many cases, as indicated by the lower curve.

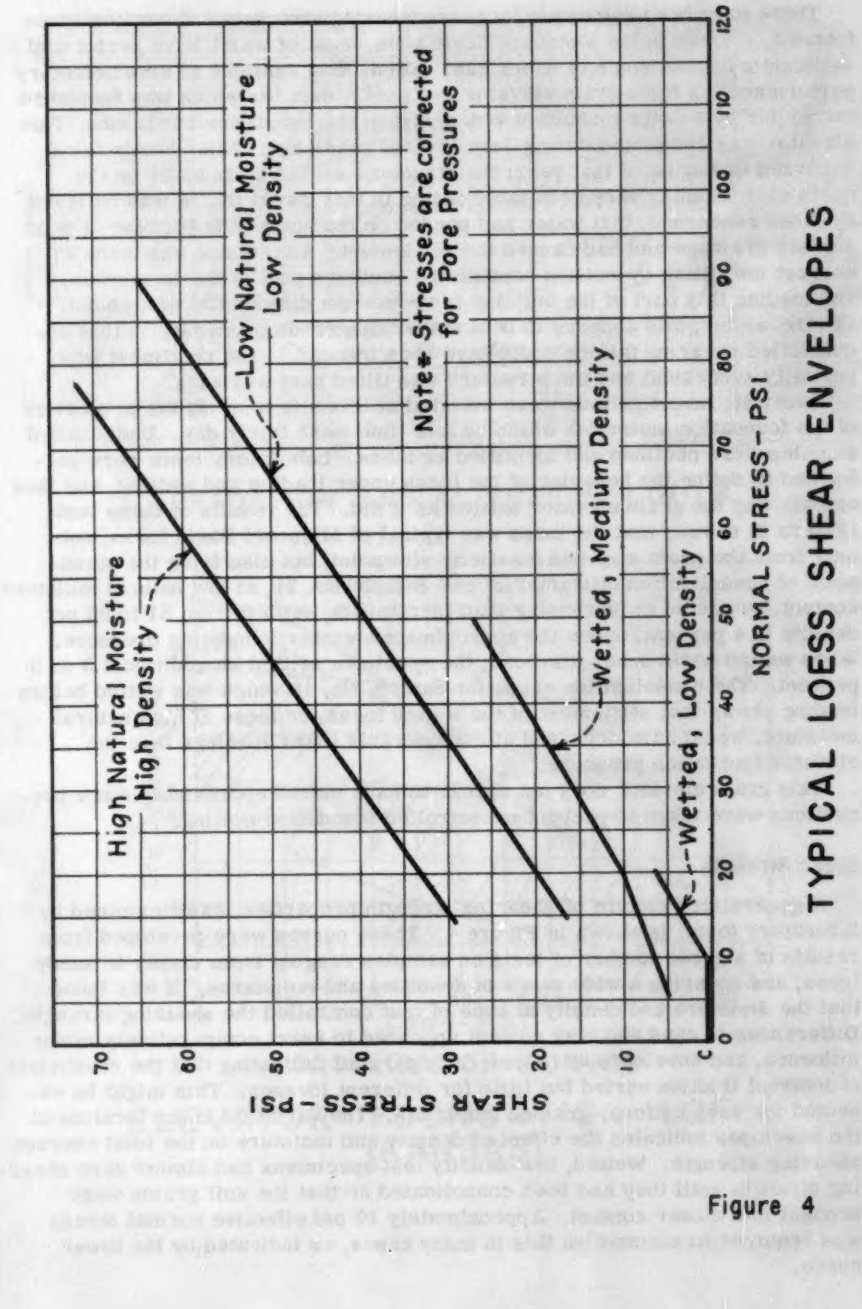

TYPICAL LOESS SHEAR ENVELOPES
NORMAL STRESS - PSI
SHEAR STRESS - PSI
High Natural Moisture
High Density
Low Natural Moisture
Low Density
Note: Stresses are corrected for Pore Pressures
Wetted, Medium Density
Wetted, Low Density
C
0
10
20
30
40
50
60
70
80
90
100
110
120

Figure 4

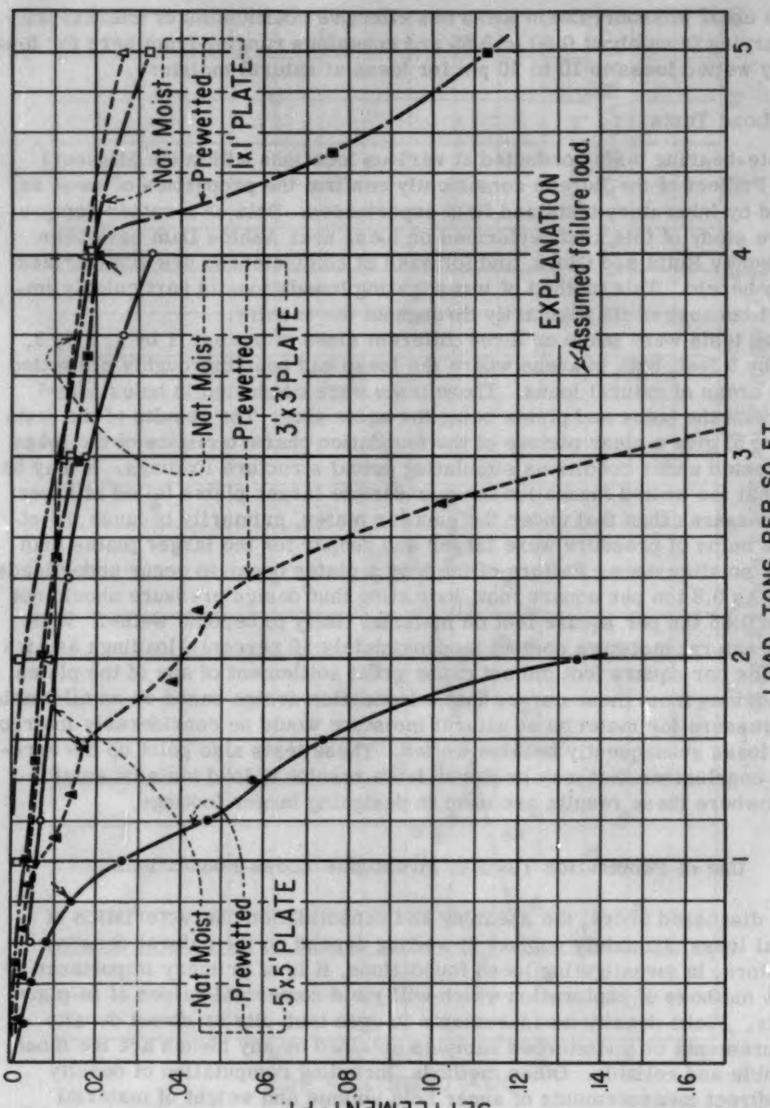


PLATE-LOAD TESTS ON NATURAL AND PREWETTED LOESS

Figure 5

The usual Missouri Basin loess has effective coefficients of internal friction varying from about 0.60 to 0.65 and cohesions ranging from zero for low-density wetted loess to 10 to 20 psi for loess at natural moisture.

Field Load Tests

Plate-bearing tests conducted at various locations within the Missouri Basin Project of the Bureau consistently confirm the properties of loess as defined by laboratory tests and field experiences. Data on a rather comprehensive study of this kind performed on loess near Ashton Dam have been reported by Holtz and Gibbs, and for sake of completeness are summarized briefly herein. This method of investigating foundations is particularly important because of its popularity throughout the country.

Load tests were made on three different sizes of plates: 1 by 1, 3 by 3, and 5 by 5 feet, both in areas where the loess had been thoroughly prewetted and in areas of natural loess. These tests were conducted in holes 5-feet deep, with the holes and plates being the same size. The results of the tests (Figure 5) give a clear picture of the foundation characteristics of the loess when tested under conditions simulating actual structure loadings. It may be seen that the wetted foundation loess under the larger plates failed at lower unit pressures than that under the smaller plates, primarily because the effective bulbs of pressure were larger and deeper for the larger plates than for the smaller ones. Failure of the 5 by 5 plates began to occur under loads as low as 0.8 ton per square foot, indicating that design pressure should not exceed 0.25 ton per square foot on material likely to become wetted. With soil at natural moisture content (approximately 10 percent), loadings as high as 5 tons per square foot did not cause great settlement of any of the plates. It is obvious from these curves that a foundation design based on an allowable soil pressure for material at natural moisture would be considerably in error if the loess subsequently became wetted. These tests also point up the erroneous conclusions that may be drawn from results of load tests on small plates where these results are used in designing larger footings.

Use of Penetration Tests to Investigate Loess Foundations

As discussed above, the shearing and consolidation characteristics of natural loess ultimately subject to wetting depend on the natural density. Therefore, in investigating loess foundations, it is of primary importance to utilize methods of exploration which will yield numerical values of in-place density. Field-density measurements in open test pits or direct density measurements on undisturbed samples obtained by any means are the most desirable and reliable. Other methods, including computation of density from direct measurements of auger hole volume and weight of material augered, have been tried but have not been greatly successful.

One of the most economical preliminary methods used by the Bureau for exploring sand foundations is by means of the "Standard Penetration Test." This test involves determination of the number of blows required to drive a 2-inch OD cylindrical sampler 1 foot into undisturbed material using a 140-pound weight dropping 30 inches. It was proposed that correlations be established so that the penetration test could be used for determining average densities of large or widely separated loess foundation areas. A program of

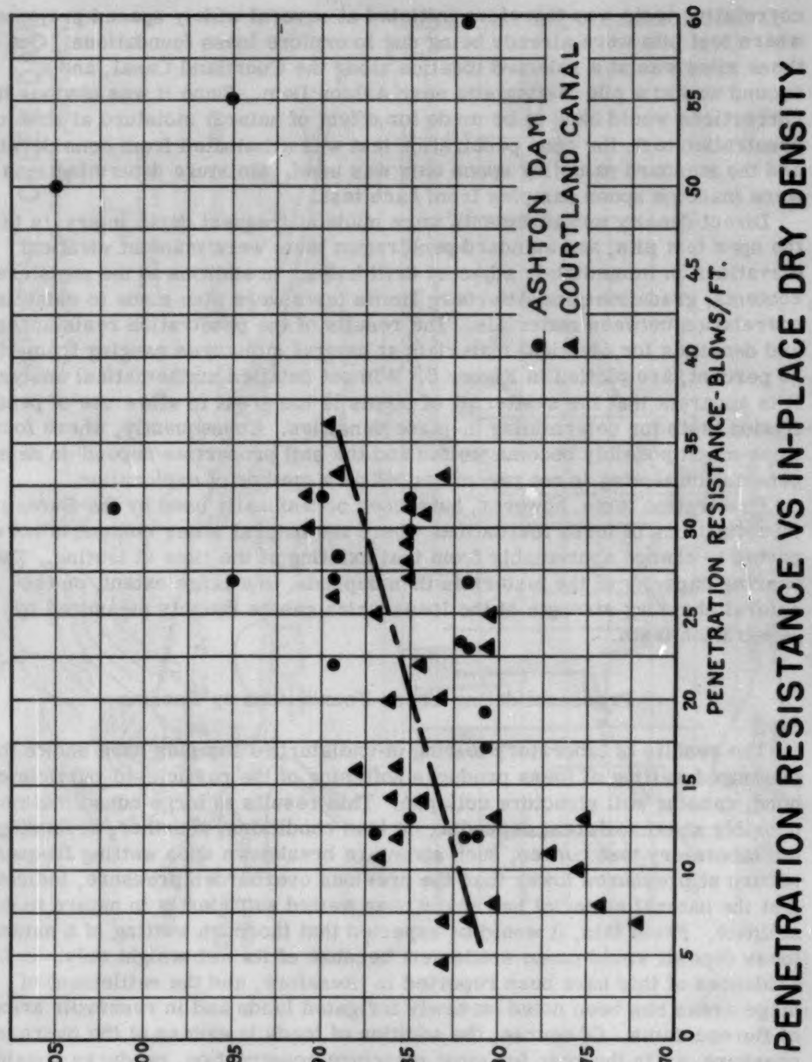

PENETRATION RESISTANCE VS IN-PLACE DRY DENSITY
● ASHTON DAM
▲ COURTLAND CANAL
PENETRATION RESISTANCE-BLOWS/FT.
0
5
10
15
20
25
30
35
40
45
50
55
60
DRY DENSITY - PCF
105
100
95
90
85
80
75
70

Figure 6

correlative tests was therefore initiated at several widely spaced projects where test pits were already being dug to explore loess foundations. One of these sites was at a selected location along the Courtland Canal, and a second was at a pile testing site near Ashton Dam. Since it was obvious that corrections would have to be made for effect of natural moisture at time of penetration test, the cone penetration test was eliminated from consideration and the standard sampling spoon only was used. Moisture determinations were made on spoon samples from each test.

Direct density measurements were made at frequent depth intervals in the open test pits, and standard penetration tests were made at identical elevations in immediately adjacent drill holes. In addition to the moisture contents, gradations and Atterberg limits tests were also made to establish correlation between materials. The results of the penetration resistances and densities for identical materials at natural moistures ranging from 10 to 15 percent, are plotted in Figure 6. Without detailed mathematical analysis, it is apparent that the scattering of points is too great to allow use of penetration tests for determining in-place densities. Consequently, where foundations might possibly become wetted and the soil properties depend on density, penetration testing is not recommended as a method of exploration.

Penetration tests, however, have been occasionally used by the Bureau in investigations of loess foundations where the natural water content is not expected to change appreciably from that existing at the time of testing. The bearing capacity of the materials then depends, to a large extent, on the natural shearing strength of the loess which can be roughly measured by penetration tests.

Preconsolidating Loess Foundations by Ponding

The results of laboratory testing of undisturbed samples have shown that prolonged wetting of loess produces softening of the particle-to-particle clay bond, causing soil structure collapse. This results in large consolidations or possibly shear failures, depending on load conditions. Further, according to the laboratory test curves, such structure breakdown upon wetting frequently occurs at pressures lower than the previous overburden pressure, indicating that the natural material has never been wetted sufficiently in nature to consolidate. From this, it would be expected that thorough wetting of a natural loess deposit would cause settlement because of its own weight only. In fact, evidences of this have been reported in literature, and the settlement of large areas has been noted on newly irrigated lands and in reservoir areas of Bureau dams. Of course, the addition of loads in excess of the overburden pressure, as is the case for most structure construction, produces considerable consolidation.

The possibility of preconsolidating loess foundations has appealed to many engineers and designers, particularly for hydraulic structures such as dams and canals where the materials will eventually become wetted. For earth dams, which exert high pressures but are rather flexible, the theory indicated that by ponding, settlement could be attained <u>during</u> construction rather than later, after completion and filling of the reservoir. For concrete canal structures, many of which are fairly rigid and can be damaged by large settlement, ponding presented a method of securing settlement <u>before</u>

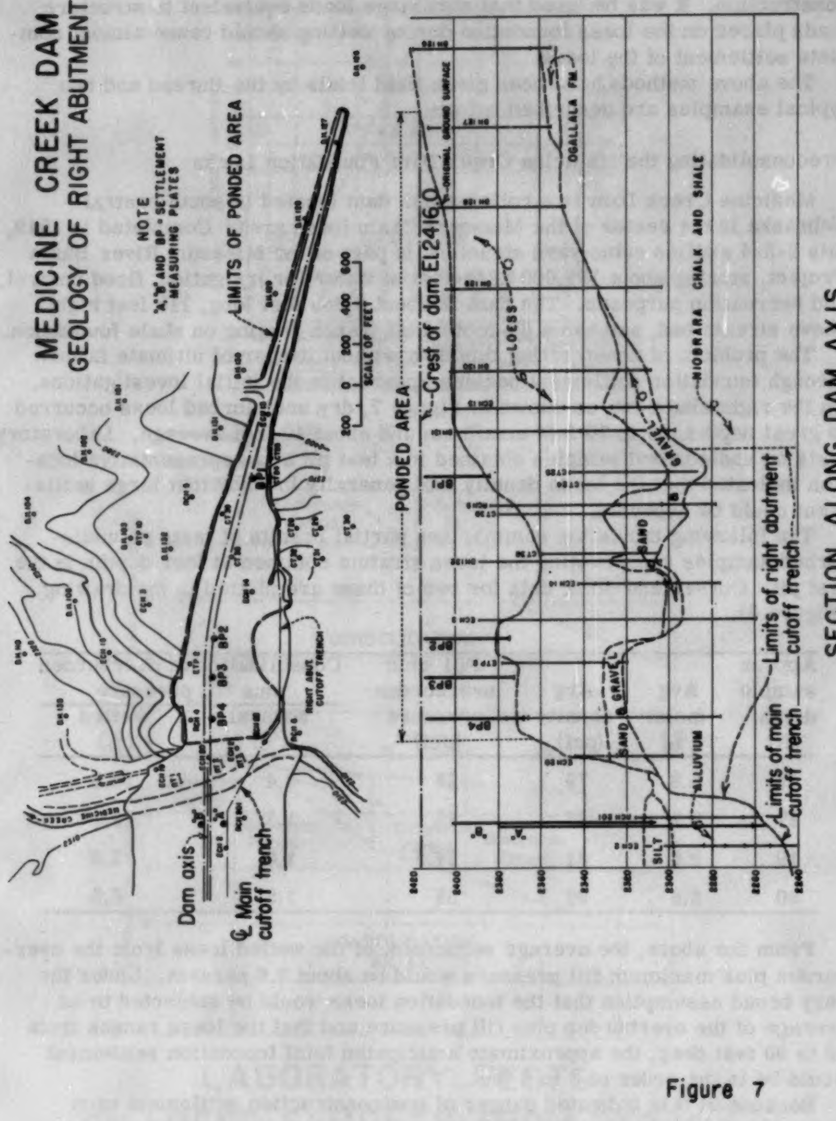

MEDICINE CREEK DAM
GEOLOGY OF RIGHT ABUTMENT
NOTE
"A","B" AND BP - SETTLEMENT MEASURING PLATES
200 0 200 400 600
SCALE OF FEET
LIMITS OF PONDED AREA
Dam axis
℄ Main cutoff trench
RIGHT ABUT. CUTOFF TRENCH
BP4
BP3
BP2
PONDED AREA
Crest of dam El2416.0
ORIGINAL GROUND SURFACE
LOESS
GRAVEL
NIOBRARA CHALK AND SHALE
OGALLALA FM.
SAND
SAND & GRAVEL
ALLUVIUM
SILT
Limits of main cutoff trench
Limits of right abutment cutoff trench
SECTION ALONG DAM AXIS

Figure 7

construction. It was believed that surcharge loads equivalent to structure loads placed on the loess foundation during wetting should cause almost complete settlement of the loess.

The above methods have been given field trials by the Bureau and two typical examples are described below:

Preconsolidating the Medicine Creek Dam Foundation Loess

Medicine Creek Dam is a rolled-earth dam located in south central Nebraska in the center of the Missouri Basin loess area. Completed in 1949, this 2-3/4 million cubic yard structure is part of the Missouri River Basin Project, storing about 196,000 acre-feet of water for irrigation, flood control, and recreation purposes. The dam is about 5,665 feet long, 115 feet high above stream bed, and has a 65-foot cutoff trench resting on shale foundation.

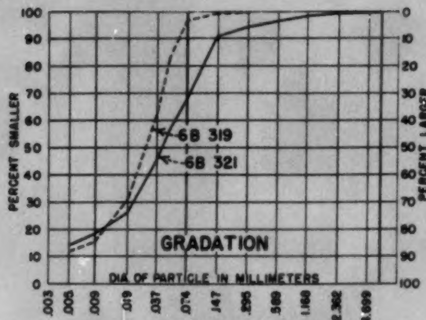
The problem of constructing this dam without danger of ultimate failure through foundation settlement became apparent in the initial investigations. On the right abutment, as shown in Figure 7, dry undisturbed loess occurred to great depths, 60 to 70 feet maximum and about 40 feet average. Laboratory tests on undisturbed samples obtained in a test pit at a representative location indicated that the loess density was generally low and that large settlement could be expected.

The following tabulation summarizes partial results of tests on undisturbed samples representing the loess stratum obtained at four depths in the test pit. Curves and other data for two of these are plotted in the drawing (Figure 8).

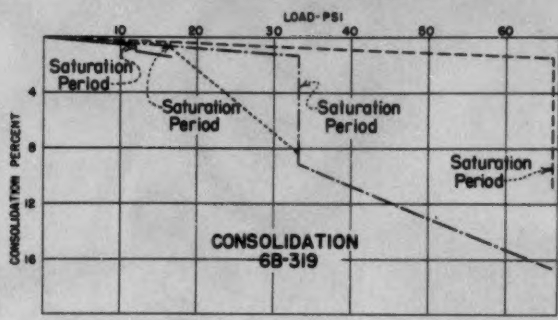
Approx sample depth (ft)	Avg moist (%)	Avg density (pcf)	Fill plus overburden pressure (psi)	Consolidation at overburden plus fill pressure	
				Natural (%)	Wetted (%)
5	8.8	79	25	8.4	10.9
17	9.7	77	33	1.3	9.1
19	9.6	81	34.5	1.0	3.9
50	6.6	92	55	1.5	6.5

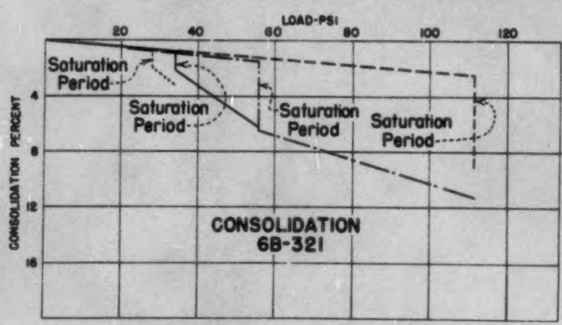
From the above, the average settlement of the wetted loess from the overburden plus maximum fill pressure would be about 7.6 percent. Under the very broad assumption that the foundation loess would be subjected to an average of the overburden plus fill pressure and that the loess ranges from 40 to 80 feet deep, the approximate anticipated total foundation settlement would be in the order of 3 to 6 feet.

Because of this indicated danger of postconstruction settlement upon saturation by reservoir water, the foundation in this area was thoroughly wetted before fill construction by ponding and sprinkling. The photograph (Figure 9) shows the dikes and the ponds full of water. Moisture content of the loess in the critical area was raised from about 12 to 28 percent average in from 25 days to 2 months using 33 million gallons of water. Settlement measuring points were established throughout these ponded areas. It is significant that no settlement occurred in these areas from saturation alone.



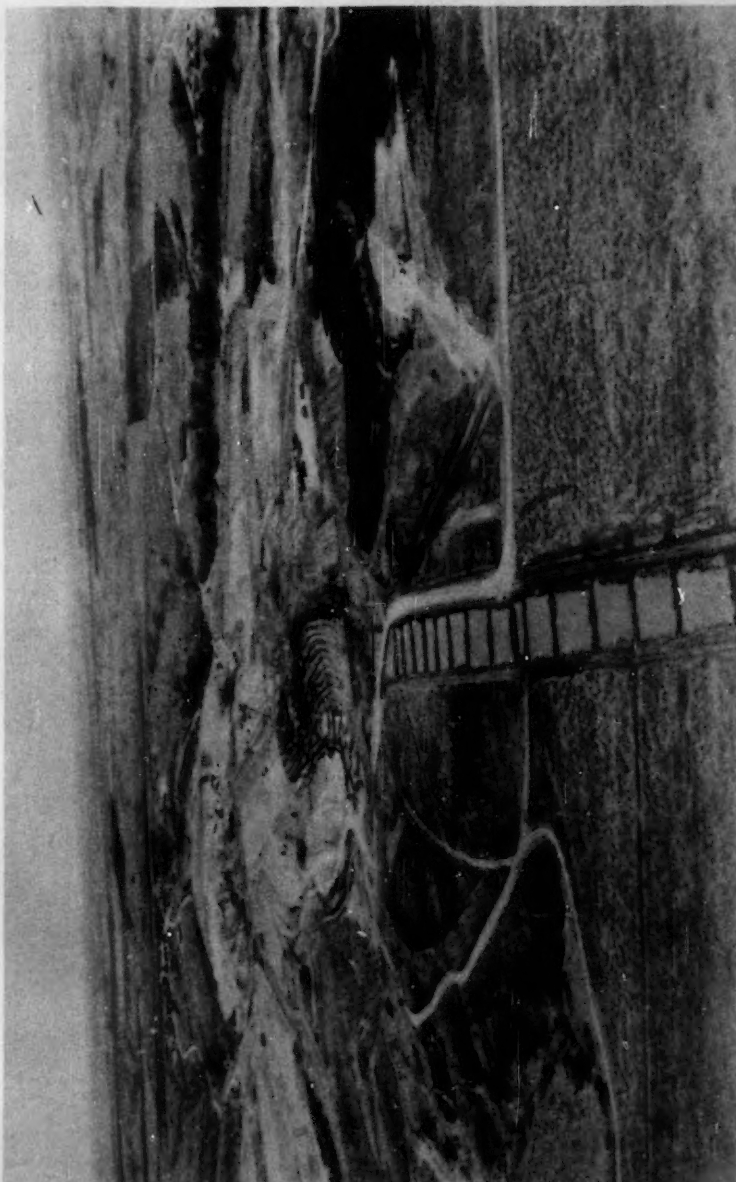
NO.	DEPTH	CLASS	DRY DENSITY	%MOIST	SPECIFIC GRAVITY
6B-319	16.5-17.6	SILTY LOAM	75 TO 79	9.5-9.8	2.68
6B-321	49.2-50.2	LOAM	89 TO 96	4.9-7.9	2.66





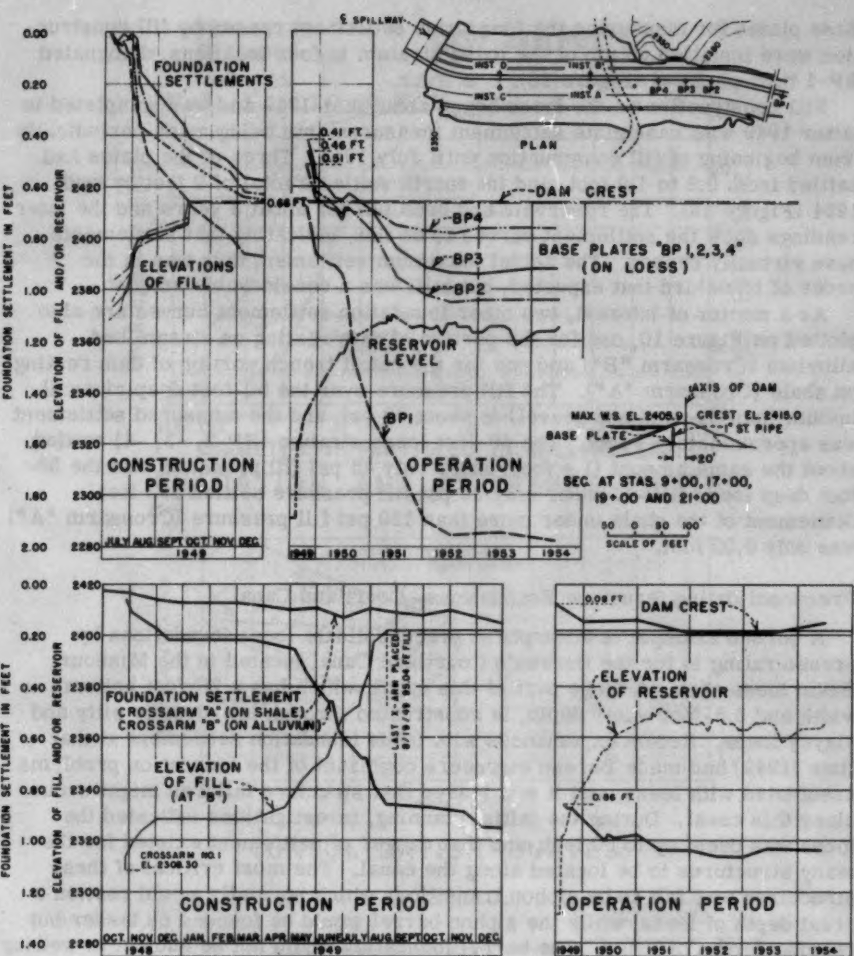
LABORATORY TESTS
ON LOESS SAMPLES FROM RTP 10
MEDICINE CREEK DAM

Figure 8



PRECONSOLIDATING LOESS FOUNDATION BY PONDING
MEDICINE CREEK DAM

Figure 9



FOUNDATION SETTLEMENT INSTALLATIONS

MEDICINE CREEK DAM

Figure 10

Base plates for measuring the foundation settlement caused by fill construction were installed on top of the loess stratum in four locations, designated BP-1 through BP-4 (Figure 10).

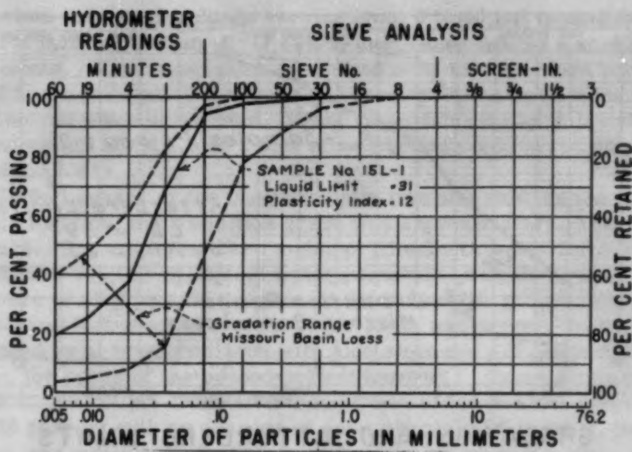
Fill construction on the loess began about mid-1949 and was completed in latter 1949 with base plate settlement measurements being made periodically from beginning of fill construction until July 1954. Three of the plates had settled from 0.8 to 1.0 foot, and the fourth settled a total of 2 feet by mid-1954 (Figure 10). The reservoir had been full for about 3 years and the later readings show the settlement curves to be flat, indicating that settlements have virtually ceased. The actual maximum settlement thus was in the order of one-third that expected, but still was a considerable amount.

As a matter of interest, two other foundation settlement curves are also plotted on Figure 10, one for the portion of dam resting on stream bed alluvium (Crossarm "B") and one for the cutoff trench portion of dam resting on shale (Crossarm "A"). The fill pressure over the 50-foot deep river alluvium (mostly sand and gravel) is about 90 psi, and the measured settlement was approximately 1 foot. The 40-foot loess stratum (BP-2, -3, -4) settled about the same amount (1 $\pm$ foot) under only 45 psi fill pressure, and the 80-foot deep loess (BP-1) under only 35 psi fill pressure settled 2 $\pm$ feet. Settlement of the shale under more than 150 psi fill pressure (Crossarm "A") was only 0.02 foot.

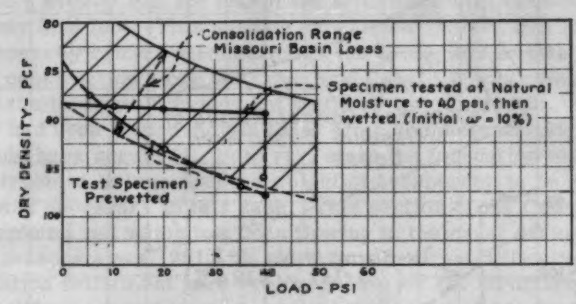
Preconsolidating Structure Foundations—Courtland Canal

A second example of attempts to preconsolidate loess foundations by presaturating is for the Bureau's Courtland Canal located in the Missouri Basin loess area. A large part of this canal, which has a 26-foot bottom width and 8.5-foot water depth, is constructed through low-density silty and clayey loess. Recent experiences with loess foundation settlement at that time (1949) had made Bureau engineers cognizant of the foundation problems associated with loess, and it was feared that structure failures might occur along this canal. During the initial planning, investigations indicated the loess was deep, up to 50 feet, and that danger of settlement existed for the many structures to be located along the canal. The most critical of these structures was felt to be siphon transitions which normally would rest on a great depth of loess, while the siphon barrel would be founded on lesser but varying depths of loess. The barrel foundation would not be subject to wetting as would the transition foundation. It was expected that water from the canal would thoroughly saturate the loess and cause differential settlement between the transition and the barrel.

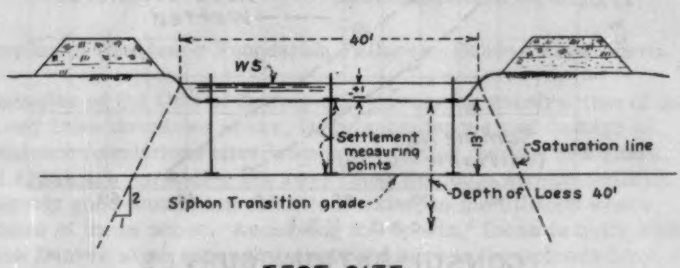
Thorough investigations were made of the loess along the critical section of alignment, including drilling, undisturbed sampling, and laboratory testing of undisturbed samples. Results of these tests confirmed that there might be cause for concern and that field tests should be performed to see if preconsolidation of the transition foundations should be attempted prior to construction. One site was selected for field trials. At this site a transition would eventually be located and unfavorable conditions were found, as indicated, by the exploration and testing. The results of the laboratory testing showed that the loess at this site was medium plastic, contained about 95 percent finer than the No. 200 sieve size, and fell well within the gradation range for Missouri Basin loess (Figure 11). Consolidation test specimens at natural



GRADATION AND ATTERBERG LIMITS



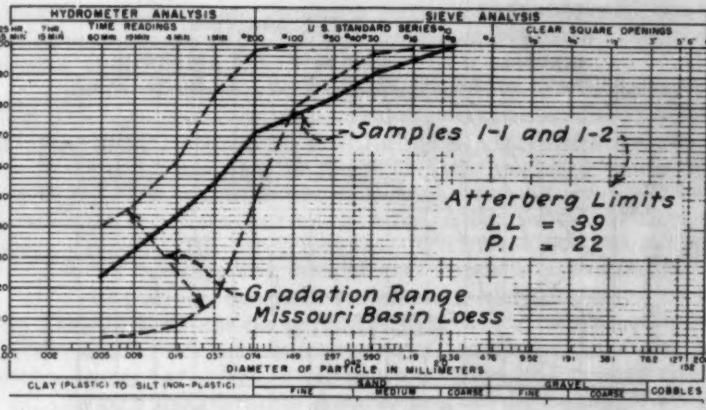
CONSOLIDATION CURVES



TEST SITE

PRECONSOLIDATION OF LOESS FOUNDATION BY SATURATION—COURTLAND CANAL

Figure 11



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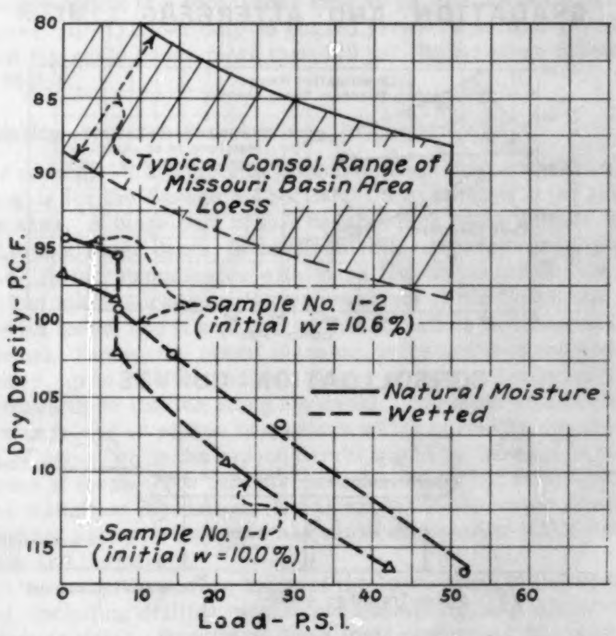


CONSOLIDATION CURVES

DENVER AREA LOESS — RESIDENCE NO. 1

Figure 12

moisture content subjected to loads varying from 0 to 40 psi consolidated very little, but upon saturation under 40 psi load, consolidated a considerable additional amount. Other specimens which had been wetted before testing consolidated to approximately the same end density as the specimens which had been saturated while under load. From these data, it was calculated that under its own weight about 1.5 or 2 feet of settlement could occur upon field saturation of the loess.

An area 40 feet square at the transition site was selected for the field tests. At this location, the transition grade was approximately 13 feet below ground surface. The area was diked and kept filled with water for 3 weeks, during which time 330,000 gallons of water permeated into the foundation. Test holes were drilled outside the area to determine the progress of saturation. It was found that good saturation of the loess was accomplished within an area bounded by a truncated cone with side slopes of 1/2 horizontal to 1 vertical from the edges of the ponded area (Figure 11). Tests showed that the moisture content of the loess after completion of the ponding ranged from about 20 to 28 percent with an average of approximately 24 percent, indicating that the loess was sufficiently wetted to cause breakdown of the loess structure. However, elevation measurements made on the various settlement measuring points showed that the maximum settlement which occurred amounted to only 0.02 foot. This ponding experiment proved that saturation would not necessarily cause consolidation of the loess, and so this plan was not utilized in construction along the Courtland Canal. It was concluded, in this case, that the loess may or may not settle by its own weight. No doubt, if a surcharge had been placed on this loess after complete wetting, some settlement would have occurred. However, since the foundation loading of such a canal structure is less than the weight of overburden to be removed, no surcharge was necessary in this case. This section of the Courtland Canal was completed and water has been flowing in the canal during most of the irrigation seasons since 1951. To date, no significant difficulties because of foundation settlement have been reported for the structures along the canal. If settlements had begun to occur, it was planned to densify the specific structure foundations by the method of silt injection.[3]

Examples of Residence Foundation Failures—Denver Area Loess

With expansion of the City of Denver and increasing construction of subdivisions away from downtown areas, large subsidences and damage of Denver residence foundations have been noted during the last few years. Residential areas are spreading out away from the main stream valleys, where relatively good foundation conditions exist, to the hilltops where major deposits of loess occur. According to reports,[4] loess is quite widespread in the Denver area, extending eastward across the uplands from each

3. "The Stabilization of Soil by the Silt Injection Method for Preventing the Settlement of Hydraulic Structures and the Preventing of Leakage from Canals," presented before the Nebraska Irrigation Association, Hastings, Nebraska, December 7-8, 1950.
4. "Pleistocene and Recent Deposits in the Denver Area, Colorado," Geological Survey Bulletin 996-C.

of the main valleys. These deposits are said to be Wisconsin age loess, sandy near the valleys and finer in texture, sandy silt to silt to the eastward. Weathering of the silt has developed some clay, which has been identified petrographically (in only a few samples) as chiefly montmorillonite. Much of the loess is marked by occurrence of lime in the form of small, separated nodules, but some of it is not.

Fairly complete investigations have been made by soils consultants of residence foundation failures in loess and of the detailed soils properties of the loess. Two of these have been selected as typical examples and are described below. Comparisons of the soil properties of the specific Denver area loesses have been made to those of the Missouri River Basin loess.

Denver Residence No. 1

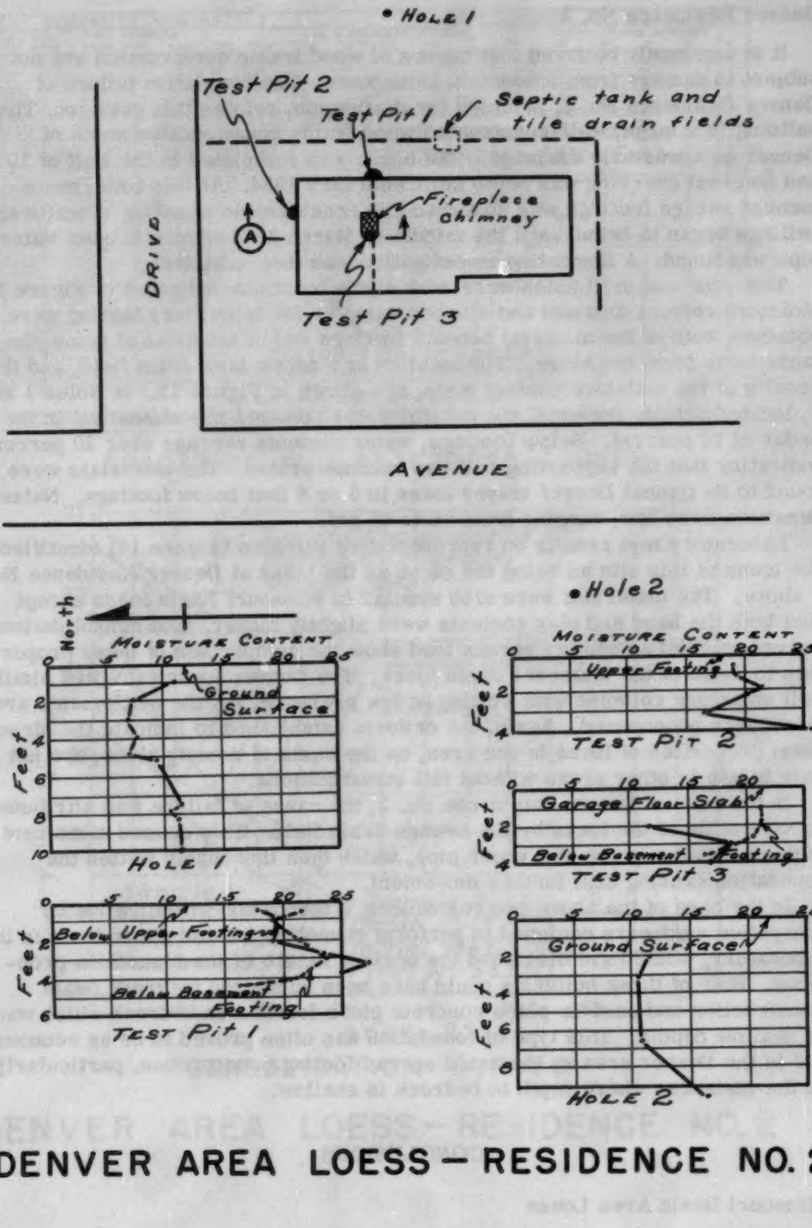
This residence was built just outside the southeast city limits, on the heights east of, and overlooking the South Platte River. It is constructed of flagstone (sandstone blocks) and is founded on spread footings. Initial cost of the house and appurtenant structures was in excess of $100,000.

In 1953, soils consultants were called to study foundation conditions since extreme damage had occurred because of footing movements. A chimney on the west side had moved outward 4 to 5 inches; horizontal and vertical movements throughout the remainder of the house caused cracking of walls and ceilings, displacement of floors, distortion of door frames, and other serious damage.

Foundation drilling disclosed that up to 7 or 8 feet of loess overlying Denver formation occurred below footings. Moisture contents of samples from beneath footings were high, ranging from 19.3 to 27.8 percent as compared to natural moisture contents of about 10 percent for unwetted materials a short distance away.

Undisturbed samples of the unwetted loess were obtained and tested in the laboratory. Figure 12 shows typical results of these tests. The grain size distribution was found to be somewhat similar to that for Missouri Basin loess, although there was a slightly higher content of medium sand, and the samples were considerably more plastic. Undisturbed specimens subjected to estimated footing pressures in the laboratory consolidated only slightly. Upon saturation at this pressure, large additional settlement occurred and with increasing applied pressures still more settlement occurred. The plot of density versus load (Figure 12) shows how the specimens changed in density throughout the test, and how these specimens consolidated greater amounts for equal load increments than those of the Missouri Basin loess. It may be seen that the broad criteria for settlement of loess based on numerical density, as discussed earlier for Missouri Basin loess, would not be applicable here.

The basic cause of the building settlement is made quite obvious by the laboratory test curves and investigation of other conditions. At natural water content the loess supported the house adequately. Watering of the lawn which was planted around the house eventually raised the moisture content and some settlement occurred. This settlement apparently caused rupture of a water pipe which thereupon thoroughly saturated the foundation materials, causing complete collapse of the soil structure and subsequent large footing movements. There were evidences of consolidation and shear failures of the loess since both vertical and horizontal movements were noted.



DENVER AREA LOESS — RESIDENCE NO. 2

Figure 13

Denver Residence No. 2

It is commonly believed that houses of wood frame construction are not subject to damage from foundation settlement. The foundation failure of Denver Residence No. 2, selected for discussion, refutes this premise. This building is a large, well-constructed wood frame house located south of Denver on a westerly drainage. The house was completed in the Fall of 1953, and no great cracking was noted until February 1954. At this time, movement of garage footings was observed and considerable cracking in walls and ceilings began to occur until the middle of March 1954 when a broken water pipe was found. A foundation investigation was then initiated.

Test pits and drill holes were sunk at the locations indicated in Figure 13. Moisture content data and undisturbed samples for laboratory testing were obtained, both of the material beneath footings and of material at some distance away from the house. The location of a septic tank drain field, and the results of the moisture content tests, are shown in Figure 13. In Holes 1 and 2, located outside the area, the natural water contents are almost all in the order of 10 percent. Below footings, water contents average over 20 percent indicating that the supporting soil has become wetted. The materials were found to be typical Denver clayey loess to 5 or 6 feet below footings. Natural densities were low, ranging from 76 to 95 pcf.

Laboratory test results on representative samples (Figure 14) identified the loess at this site as being the same as the loess at Denver Residence No. 1 above. The materials were also similar to Missouri Basin loess except that both the sand and clay contents were slightly higher. The consolidation curves, plotted as density versus load show the comparison of these properties to those of the Missouri Basin loess. The Denver loess exhibited similar soil structure collapse with wetting at low pressure, but the settlements are even more pronounced. Again, the criteria established to indicate the structural properties of loess in one area, on the basis of density alone, are not safe to use in other areas without full investigations.

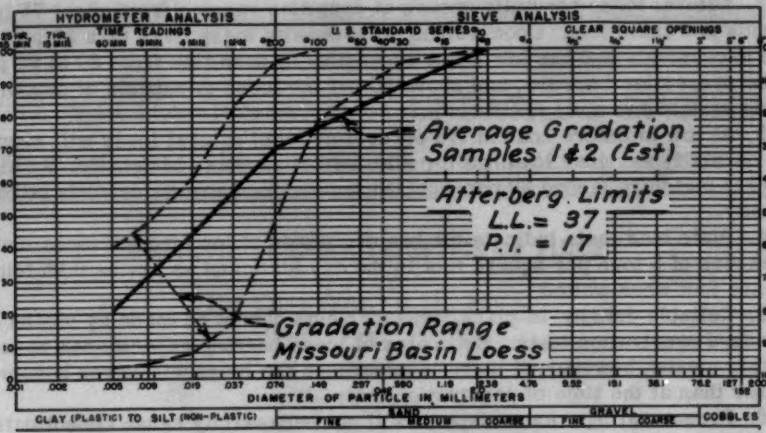
In the case of Denver Residence No. 2, the cause of failure was attributed to saturation of the loess by the sewage drain field. This caused movement with resultant rupture of a water pipe, which then thoroughly wetted the foundation causing still further movement.

In the case of the above two residences, a foundation investigation by competent engineers equipped to perform consolidation and shear tests in the laboratory, would have disclosed the serious nature of the foundation problems. Both of these buildings could have been supported by grade beam construction and cast-in-place concrete piers founded on bedrock which was at shallow depths. This type of foundation has often proved to be as economical in the Denver area as the usual spread footing construction, particularly in the instances where depth to bedrock is shallow.

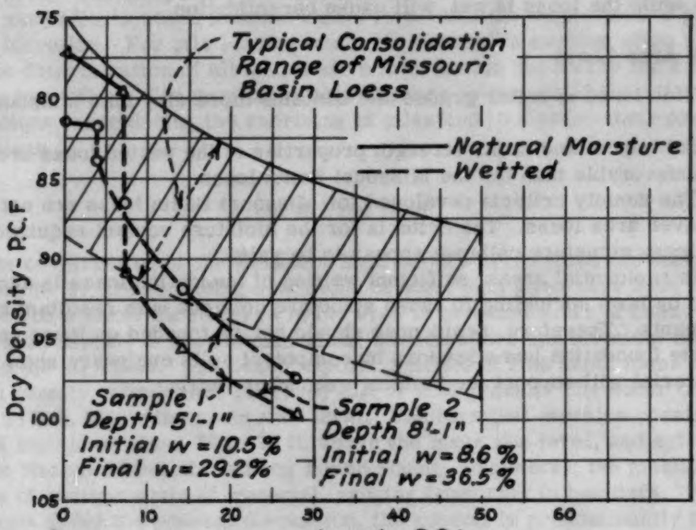
CONCLUSIONS

Missouri Basin Area Loess

1. Loess is composed primarily of silt-sized particles bonded together by small amounts of montmorillonite clay to form a typical, porous, open structure.



GRADATION AND ATTERBERG LIMITS



CONSOLIDATION CURVES

DENVER AREA LOESS—RESIDENCE NO. 2

Figure 14

2. Natural loess normally occurs at densities ranging from about 75 to 95 pounds per cubic foot. Upon wetting, low-density loess (less than 80 pcf) settles excessively and has low shear strength. At densities from 80 to 90 pcf, these properties improve, and above 90 pcf the loess is capable of supporting loads assigned to soil.

3. At low moisture content (15 percent or less) natural loess will support the normally assigned loadings for silty soil regardless of density. At high natural moisture (above 20 percent) the supporting capacity depends on the density.

4. Plate load tests indicate the bearing power of low-density loess may be in excess of 5 tons per square foot in the dry state and as low as 0.25 ton per square foot when wetted.

5. Reliable density-in-place measurements cannot be made by means of standard penetration tests. Estimates of loess bearing capacity can be made by this means, however, if there is assurance the loess will never become wetter than at the time of penetration tests.

6. Presaturating a loess deposit will not necessarily cause consolidation by the weight of the loess stratum only except perhaps when the density is very low. An external load, applied by embankment construction or other means while the loess is wet, will cause consolidation.

Denver Area Loess

7. This loess is better graded and contains more clay than Missouri Basin loess.

8. Settlement and shear strength properties of the wetted loess are even more unfavorable than for the Missouri Basin loess.

9. The density criteria developed for Missouri Basin loess are not valid for Denver area loess. The criteria for the moisture content required to cause loess structure collapse appear to be valid.

10. In residential areas, sufficient wetting of foundation loess is accomplished by lawn sprinkling to cause structure collapse with resultant great settlements. Therefore, residences should not be founded on loess unless complete foundation investigations by competent soils engineers show that the material will support the building even after wetting.

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Proceedings of the American Society of Civil Engineers

REDRIVING CHARACTERISTICS OF PILES

Nai-Chen Yang,[1] A.M., ASCE
(Proc. Paper 1026)

SYNOPSIS

In evaluating the bearing capacity of a pile, the present technique still can be improved. The temporary stress adjustments in the subsoil after pile driving have effects which to some extent invalidate the application of pile driving formulas. For pile loading tests, the excessive expense often prevents the determination of all quantitative information necessary for conclusive interpretation. The purpose of this paper is to present the results of observations made during the redriving of piles and to discuss their possible applications.

Piles in Fine Sand

At the construction of the Prospect Expressway in the Gowanus section of Brooklyn, N. Y., the redriving conditions of steel pipe piles have been observed. The subsoil encountered in the area is predominantly of sandy material. Beach deposit is encountered along the sea shore and terminal moraine forms to the high land. The beach deposit consists of fine sand, loose to medium density, with lenses of clayey silt or fine gravel. The water table is about 2 to 8 ft. below the mean sea level. The terminal moraine constitutes a belt of high land, about 30 to 70 ft. above the mean sea level, and extends from the Narrows toward the Long Island Sound. In general, the glacial till consists of various sorts of material, ranging from clay to boulders. For the till deposit under the present discussion, the subsoil is predominantly of fine sand with a trace of gravel and boulders. The relative density of the sand deposits are indicated in Fig. 1 by the "standard penetration" which is the number of hammer blows per foot on the sampling spoon. The ground water table in this area is about 10 to 20 ft. above the mean sea level.

The piles under redriving test were 16"OD closed end pipe, driven by a Vulcan #0 hammer and subsequently filled with concrete. The tests were conducted during the construction of foundations for the bridge over Third

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Avenue which is located about two blocks inshore. According to boring #37, which is about 150 ft. from the bridge, the subsoil consists of loose to medium density fine sand overlain by a 10 ft. layer of fill. The pile driving started at 10 to 20 blows per ft., and increased to 30 to 60 blows when the pile tip was near the water table. A resistance of 50 to 100 blows per ft. was the average condition for driving in saturated sand. When driving resistance exceeded 100 blows per ft., the pipe shell tended to broom and the driving had to be interrupted. After trimming the pile top, driving was resumed. At the beginning of redriving, the number of hammer blows per ft. decreased to about one half of that before the intermission. Fig. 1 shows one representative driving pattern for 30 observed piles.

Another group of 58 piles were subjected to the same type of redriving test. These piles were 12"OD closed end pipe driven by a Vulcan #1 hammer. The test piles were located about half a mile inshore where terminal moraine is encountered. The redriving resistance varied from 0.4 to 0.7 of the hammer blows before intermission. Fig. 2 shows the driving pattern of three representative piles.

According to the observations, it seems that there is a general pattern for the hammer blows at redriving. After a sharp decrease at the beginning, the redriving resistance picks up gradually until it reaches the original hammer blows before intermission. Then the effect of redriving no longer exists and the pile driving resumes the original pattern as if the pile had been driven without any intermission.

Steel Piles in Silt and Clay

For the foundation of the Tappan Zee Bridge spanning the Hudson River at Tarrytown and Nyack, N. Y., piles have been used extensively. Series of loading tests and redriving observations have been made. The scope of this paper is limited to the description of redriving observations and soil conditions in brief.

The unconsolidated deposits encountered at the bridge site were predominantly of clayey and silty soils. A layer of boulder clay, about 10 to 20 ft. in thickness, overlies the bed rock which is about 300 ft. below water level at the main pier points. Presumably, the boulder clay is a ground moraine which has been loaded by the weight of glaciers as well as the overburden of subsequent sediments. Irregular pockets and lenses of fine grained soils are intermingled with boulders.

Overlying the boulder clay, the subsoil consists of a thick layer of varved clay. Its compressive strength ranges from medium to stiff, natural water contents from 20 to 38% and compressive index, C_c, from 0.12 to 0.30. According to the strength of remolded samples, the varved clay is classified as a sensitive soil.

Overlying the varved clay, a layer of very uniform fine sand of medium to dense consistency is encountered. Its thickness ranges from 10 to 25 ft. and it extends horizontally over the entire river basin between elevations 100 and 135 ft. below datum. This material is a transitional deposit between the glacial formations and more recent sediments.

The uppermost layer of the river bed consists of a thick layer, about 70 to 100 ft. of organic alluvial silt. It is a warm water deposit accumulated in recent geological time. The average thickness of its annual sedimentation is

estimated to be less than 0.1 inch which is equivalent to the thickness of lamination. Because of its richness in organic contents together with its laminated structure, the silt deposit resembles a varved clay insofar as the sensitivity is concerned. The shearing strength ranges from very soft to medium consistency, natural water contents from 50 to 65% and compression index from 0.4 to 0.7. It is a highly compressible and normally loaded deposit.

According to previous experience of driving piles in silt and clay deposits, it is known that a pile will increase its resistance against driving or pulling after an intermission of driving. In field engineering terminology, a pile showing such an increase in resistance is said to have been "frozen." In order to experiment with the actual "frozen" condition encountered in the organic silt and varved clay, field observations were made during the driving of steel H piles for the north cofferdam of Pier #168. The foundation consists of a 35 ft. diameter sheet pile cofferdam and 52 steel H piles driven to basement rock. A typical driving pattern and the boring at the center of the pier are shown in Fig. 3. The purpose of the observations was to establish the time effect on the change of hammer blows after an intermission of driving. The same driving rig, shipment of piles and crew of personnel have been employed throughout the observations. The time interval was designed to simulate the consolidation test of a soil sample. In considering the contractor's working hours, the tests were made on a group of 12 piles at a staggered time interval of 1/2, 2, 18 hours, and 1, 2, 5 and 11 days. The testing results are summarized in Table I and are shown graphically in Fig. 4.

According to the test results, there is a significant increase of hammer blows at the beginning of redriving. The subsequent hammer blows tend to decrease and merge with the original driving pattern after 2 to 6 ft. of penetration. The relation between time interval and increasing hammer blows at redriving resembles a time consolidation curve. It seems that the consolidation of the surrounding soils has a certain role in connection with the redriving resistance of a pile. It is interesting to note that the redriving resistance increases significantly for piles in a clay-silt deposit but it decreases in fine sand.

Timber Piles in Silt

For the approach spans of the Tappan Zee Bridge, timber piles have been used. Loading and driving tests were made at various sections of the westerly approach. All test piles were subsequently retrieved. The soil condition encountered by the timber piles is identical to the organic alluvial silt previously described. Although the final driving resistance was only 2 or 3 blows per ft. for a 5-kip hammer, the timber piles could hardly be retrieved by a floating rig having a lifting power of more than 50 tons. It was found that a pile had to be redriven in advance of pulling. The redriving resistance usually started at 30 to 60 blows per ft. and decreased to about 10 to 20 blows per ft. after a few feet of penetration. Then, the timber piles could be pulled out at a resistance of about 25 to 50 tons. A typical driving pattern and nearby soil boring are shown in Fig. 5. The observed redriving resistance, and time of intermission are summarized in Table II. For the purpose of comparison, redriving information is incorporated for a group of steel H piles driven in the same organic silt deposit. Although the average time of intermission was

about 40 days shorter for steel H piles, the observed results, as shown in Fig. 4, will suggest that the redriving resistance of a timber pile is apparently greater than that of a steel pile. The higher resistance of timber piles is primarily due to the pervious characteristics of the pile material.

Review of Pile Driving Formulas

For determining the bearing capacity of a single pile, the use of pile driving formulas has been debated for some years. Yet, their application is still practiced on many piling jobs. Presumably, there is no substitute which can match the simplicity of pile driving formulas. The discrepancy between the test load at failure and the ultimate resistance computed by various pile driving formulas has been compiled by Mr. R. D. Chellis,[2] M., ASCE. He concluded that ". . . . . The best and safest range of values . . . . . appears to be obtained from the (Hiley's) formulas . . . . . ," which can be simplified as

$$Q = \frac{EWH}{s + \frac{1}{2}c} \qquad (1)$$

where, Q is the ultimate resistance of a pile at driving, s is the penetration for each blow, c is the range of elastic deformations, WH is the impulse of each hammer blow, and E is the efficiency of driving. From the point of view of a field engineer, Equation (1) is not only simple in its fundamental mechanics but it is also practical for job application. However, the value of E and c are not known by observing the hammer blows at driving. Any arbitrary change in their values will result in a deviation of the pile driving formulas.

Furthermore, the bearing capacity computed by a pile driving formula represents only the resistance at the instant of driving. The effect of stress adjustment which will take place in the subsoil after the intermission, is not reflected by any pile formula. By interpreting Mr. Chellis' compilation,[2] there are definite indications that the magnitude of the discrepancies resulting from the use of the Hiley formula are more or less related to the soil condition encountered by a pile (see Fig. 6). For piles in clay-silty soils, the test load at failure is much greater than the ultimate resistance by the Hiley formula. On the other hand, for piles in fine sand, the failure load is relatively smaller. Because there is no provision for subsoil conditions, the Hiley formula is too conservative for clay-silty soils but rather risky for piles in fine sand.

In Fig. 6, the average test load at failure in terms of the ultimate resistance computed by the Hiley formulas is 0.9 for piles in fine sand and 1.6 for clay-silty deposit. According to the observations on steel piles, the hammer blows at the beginning of redriving, in terms of final hammer blows before intermission, reduces to one half for piles in fine sand but increases to 10 to 12 times for clay-silty soils. For both the test load at failure and the hammer blows at redriving, the fluctuations are practically identical on a logarithm plot but different in scale (see Fig. 6). This result suggests the possibility of a close relationship between the test load at failure and the hammer blows at redriving.

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2. Chellis, R. D., (1951), Pile Foundations, pp. 551 - 570.

Hypothesis on Pile Resistance

At the time of driving, the subsoil around a pile is subjected to a rapid change in volume. The displacement of soil cannot take place as rapidly as the intrusion of pile. The driving impulse is temporarily carried by the porewater and a zone of excessive porewater pressure is formed around the pile.

After the recess of driving, the excessive porewater pressure has to be dissipated in order to reach a state of equilibrium in the subsoil. The decrease of porewater pressure is usually associated with (1) the drainage of porewater by consolidation and (2) the plastic movement of subsoil, which not only relaxes the source of pressure but also results in the heaving and lateral movement of the ground. Hence, the adjustment of porewater pressure has a decisive effect on the resistance of a pile. For piling in sand deposits, the excessive porewater pressure at driving reduces the effective confining pressure around the pile. Because of its low permeability, as well as low plasticity, the porewater in a fine dense sand acts like the fluid in a hydraulic jack. The driving impulse is, therefore, temporarily carried by the porewater at the pile tip. If the intermission of driving is long enough to allow the adjustment of porewater, there will be no excessive pressure at the beginning of redriving and the point resistance is likely to be significantly decreased. For soft clay deposits, the natural shearing strength is usually a function of the consolidation pressure. The skin friction of a pile, therefore, depends on the consolidation of the surrounding soils. During pile driving, the excessive porewater pressure, together with the disturbance of the subsoils, prevent the full mobilization of the natural shearing strength. Moreover, the plastic movement of a soft subsoil minimizes the significance of excessive porewater pressure at the pile tip. A very easy driving is always anticipated for piling in soft clay. During an intermission in driving, the decrease of porewater pressure is associated with an additional consolidation of the soil around a pile and the disturbed soil regains its shearing strength by thixotropic effects. A relatively solidified soil shell is formed on the pile surface and the penetration at redriving does not take place along the pile surface but at some place where the newly consolidated soil meets the original one. The resistance at redriving can be many times as great as the resistance before intermission.

The rate of increase in redriving resistance depends primarily on the condition of consolidation. For timber piles, the pervious characteristics of pile material provides an additional drainage surface for the subsoil. The soil around a timber pile will consolidate to a more extensive degree than that around a steel pile. The redriving resistance is, therefore, much higher for timber piles than for steel ones. As the redriving goes on, the excessive disturbance and the growth of porewater pressure will cause a loss of resistance. The hammer blows will thus decrease gradually and merge into the same driving pattern as before intermission.

For piles subjected to loading tests or under the load of a structure, there is no excessive porewater pressure in the subsoil. It is obvious that the test load at failure bears no relation to the final driving resistance computed by any pile driving formula. At the beginning of redriving, the porewater condition is likely to be identical to the condition during a loading test. It seems that the actual bearing capacity of a pile can be more rationally expressed in terms of its redriving resistance, in which, the effect of temporary stress adjustment has been incorporated, than by the resistance during initial driving.

Loading Tests and Interpretations

In Fig. 7, a typical driving pattern is shown for the steel H piles in the soft alluvial silt and fine dense sand overlying medium to stiff varved clay. The pile resistance is analyzed in Table III. Because of its purpose for demonstration only, the analysis has been simplified by assuming (1) that the skin friction and point resistance increase proportionately with the hammer blows at El.-120' and El.-172' respectively; (2) that the rates of changes, a and b, are identical at El.-136' and El.-172'; and (3) that the number of hammer blows is adjusted by the factor $\frac{s}{s + \frac{1}{2}c}$ in which s and c are the penetration and the elastic deformation as directly measured in the field. The adjusted hammer blows represent more precisely the change of pile resistance. In Table III, the analysis indicates that (1) an increase of skin friction but a decrease of point resistance has been experienced at redriving; and that (2) the skin friction is more significant at redriving than before the intermission.

In Fig. 8, the ultimate loading condition of test pile #1 is presented. The soil condition and the driving pattern are identical above El.-136' with those shown in Fig. 7. While setting the pile by its own weight plus the weight of hammer, casing and anvil, 39 kips in total, the pile slipped down from El.-25' to El.-116'. Without long delay after setting, the pile resistance commenced at 3 blows per ft. When the pile tip was at El.-130', about 5 ft. in the sand layer, the driving resistance was about 18 blows per ft. After an intermission of three days, the resistance at the beginning of redriving was about 28 blows per ft. The same crews and driving rig were used in the operation. The elastic deformation, c, and the actual penetration, s, were measured directly on the pile surface about six feet below its top. At the beginning of redriving, c was 0.28" and s was 0.45". Immediately after the setting of pile, the resistance is known to be the total weight of 39 kips. By using the same driving rig, the driving efficiency will not change appreciably. Hence, the resistance of a pile will be

at the beginning, $$Q_o = \frac{EWH}{s_o + \frac{1}{2}c} \qquad (1A)$$

at subsequent driving, $$Q = \frac{EWH}{s + \frac{1}{2}c} \qquad (1B)$$

By eliminating the term expressing the driving efficiency, E, and also considering the fact that the value of c_o is negligible in comparison with the value of s_o, the Equation (1B) can be expressed as

$$Q = \frac{s_o}{s + \frac{1}{2}c} \; Q_o \qquad (2)$$

in which, Q_o is known to be equivalent to the total weight for setting the pile and s_o is the initial penetration. By using Equation (2), the ultimate resistance of test pile No. 1 is computed to be about 130 tons at the beginning of redriving. In the loading test, the load at failure was about 120 tons (see Fig. 8). By adding the weight of the H pile, about 8 tons, the total load at failure comes out very close to the ultimate resistance of redriving.

According to the load test pile No. 3, (see Fig. 5), the pulling resistance was observed to be about 25 tons for a final redriving resistance of 11 blows

per ft. At the beginning of driving, the resistance was 3 blows per ft. The total weight for setting the pile was 11 kips. According to Equation (2), the ultimate resistance was estimated to be about 20 tons at 11 blows per ft. Because there was a brief intermission after the completion of redriving, the pulling resistance should be a little higher than the resistance at final redriving. It seems that the computed resistance of 20 tons is a fairly good estimate.

For the subsoil encountered by the test pile No. 3, the unconfined compressive strength ranges from 0.2 tsf at the uppermost portion to 1.0 tsf at a depth of about 80 ft. below the river bottom. The average shearing strength is estimated to be about 600 psf. As the shearing deformation always takes place along the weakest zone in a soil mass, the average skin friction on the pile surface should not exceed the average shearing strength of the soil unless the deformation takes place either partly or completely beyond the pile surface. For the test pile No. 3, the embedded length is about 85 ft. and the average diameter is about one foot. According to Equation (2), the ultimate resistance is computed to be about 105 tons at the beginning of redriving. The average skin friction on the pile surface is about 780 psf which exceeds the average shearing strength of the soils by about 30%. It is suggested that the shearing deformation is taking place beyond the pile surface and a soil ring has been consolidated with the pile during the intermission of driving. The soil ring sticks to the pile surface and penetrates together with the pile at the beginning of redriving. The gaining of shearing strength by additional consolidation is apparently the primary cause for the formation of a solidified soil ring.

According to known experience,[3] a reliable pile loading test should be performed after the completion of temporary stress adjustment. The loading test is always a time consuming and expensive operation. Its result may not be conclusive for designing a large foundation. For a small piling job, the total cost of construction may prohibit a loading test. As a consequence, a great deal of design criteria has to depend on the sound judgment of a foundation engineer. It is the writer's opinion that if quantitative information on redriving resistance is in agreement with the test load at failure, the Equation (2) will provide a simplified approach to evaluate the resistance of a pile. It is relatively inexpensive to run a redriving test. The redriving observations can be made on more piles than the load tests. The quantitative information obtained from these tests will be very helpful for foundation design as well as for field management.

CONCLUSION

Summarizing the above observations, it may be concluded that: (1) after an intermission in driving, the driving resistance increases considerably for piles in clay-silty deposits but decreases in fine dense sand; (2) the increase in driving resistance for a timber pile is greater than that for a steel pile; (3) in comparison with the ultimate resistance computed by the Hiley formula, the test load at failure is much higher for piles in clay silty soils but smaller in fine sand; (4) the use of arbitrary values for the efficiency of hammer blows and the actual elastic deformations at driving will cause deviations in the value of pile resistance computed by a pile driving formula; and (5) because the

3. Terzaghi & Peck (1948) Soil Mechanics in Engineering Practice, p. 466.

temporary stress adjustment in the subsoil is not incorporated in any pile driving formula, the computed ultimate resistance is conservative for piles in clay-silty soils but rather risky for those in fine sand.

According to the porewater concept on pile resistance, it is suggested that at the beginning of redriving, the porewater condition around a pile is similar to that during a loading test. The redriving resistance is, therefore, a more rational indication of the actual loading capacity of a pile. Furthermore, by introducing the driving condition immediately after the setting of pile, the use of an arbitrary value for driving efficiency can be eliminated. By measuring the penetration and elastic deformation directly in the field, the major discrepancies of a pile driving formula can be improved materially. According to the results of the loading tests, Equation (2) provides a promising means of evaluating the ultimate resistance of a single pile. Redriving observations are relatively inexpensive and it appears that such observations can be used as a substitute for loading tests.

ACKNOWLEDGMENT

The writer wishes to express his gratitude to Capt. Emil H. Praeger, M., ASCE, for his valuable guidance on this research project. Gratitude is also extended to Mr. E. J. Herkovic for his constructive discussion in arranging the tests.

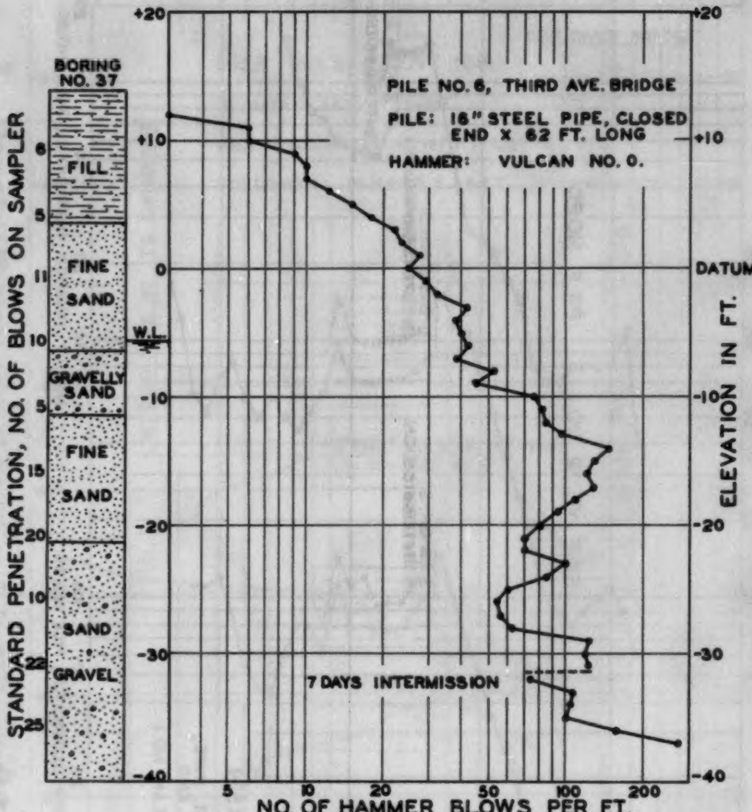

Fig. 1 Driving Pattern of Steel Pipe Pile in Fine Sand

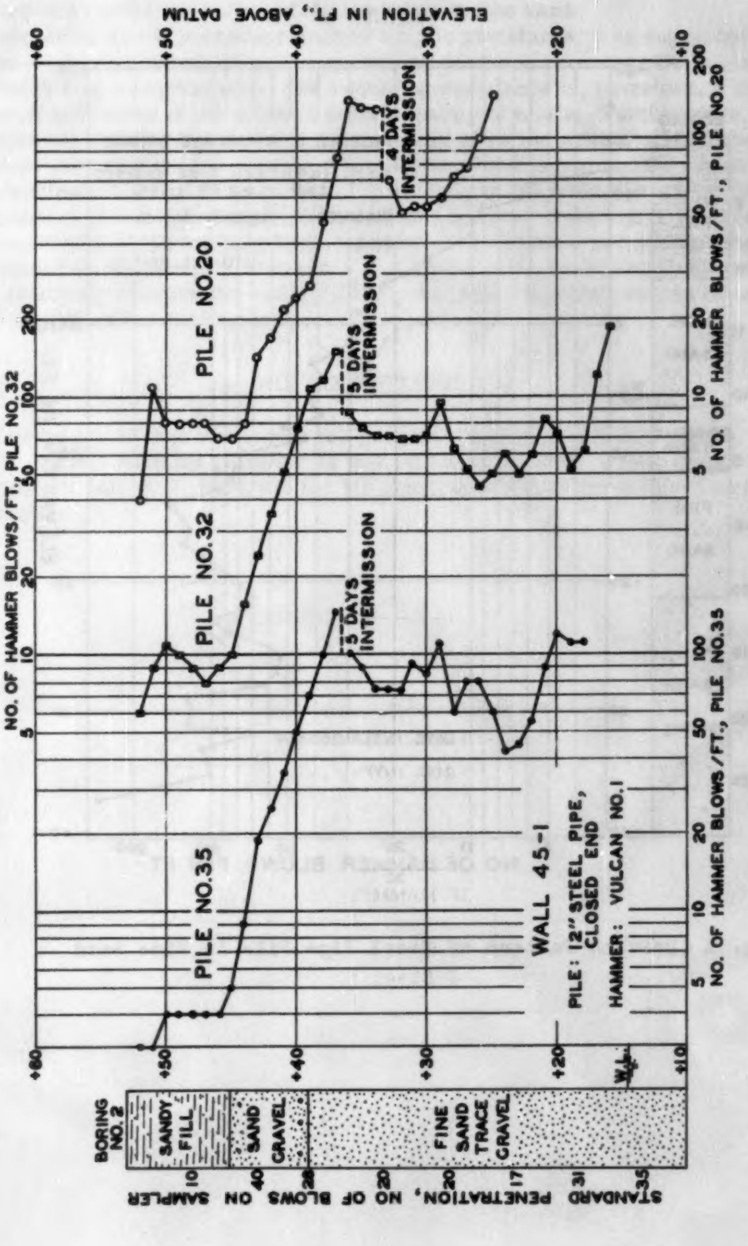

Fig. 2 Driving Pattern of Steel Pipe Piles in Fine Sandy Till Deposit

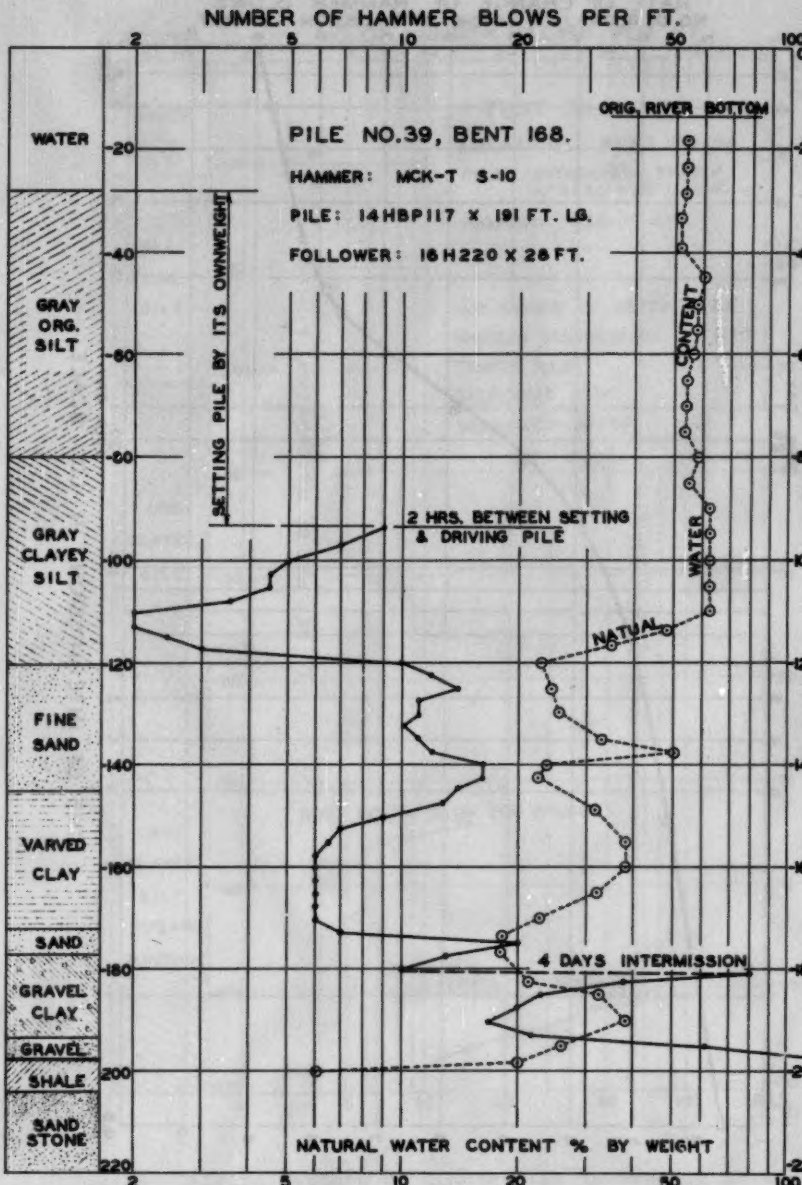

Fig. 3 Typical Subsoil Condition & Driving Pattern of Steel H Piles

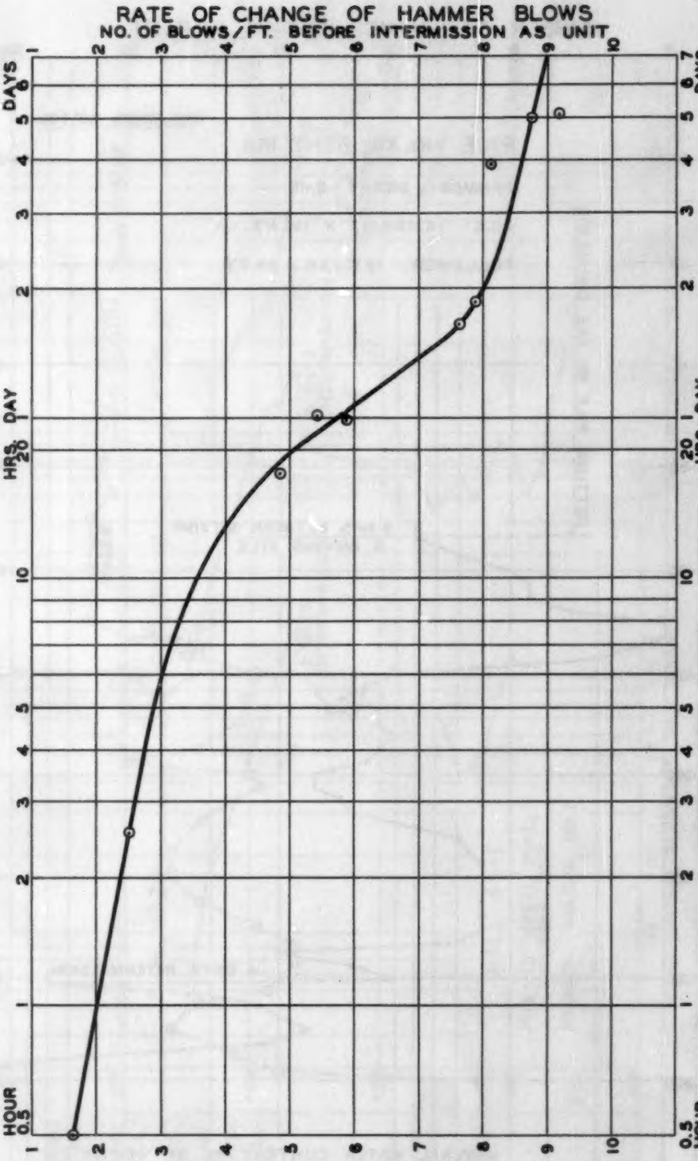

Fig.4 Observed Relation between Time of Intermission and Change of Hammer Blows at Redriving of H Piles

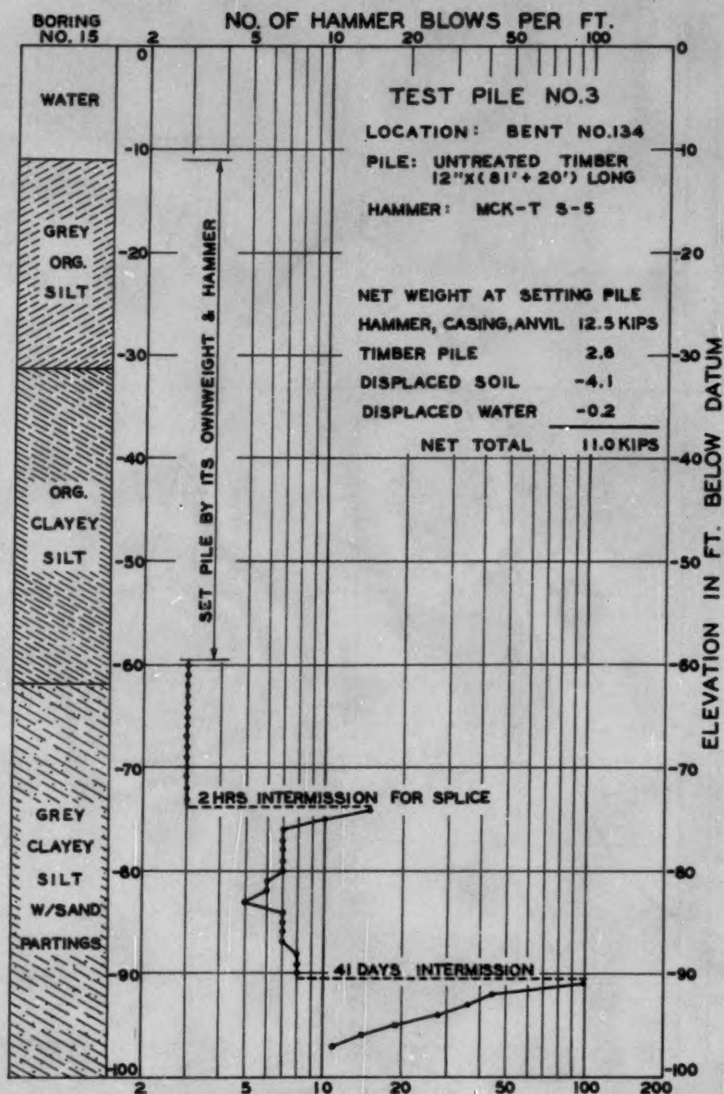

Fig. 5 Driving Pattern of Timber Pile in Organic Silt

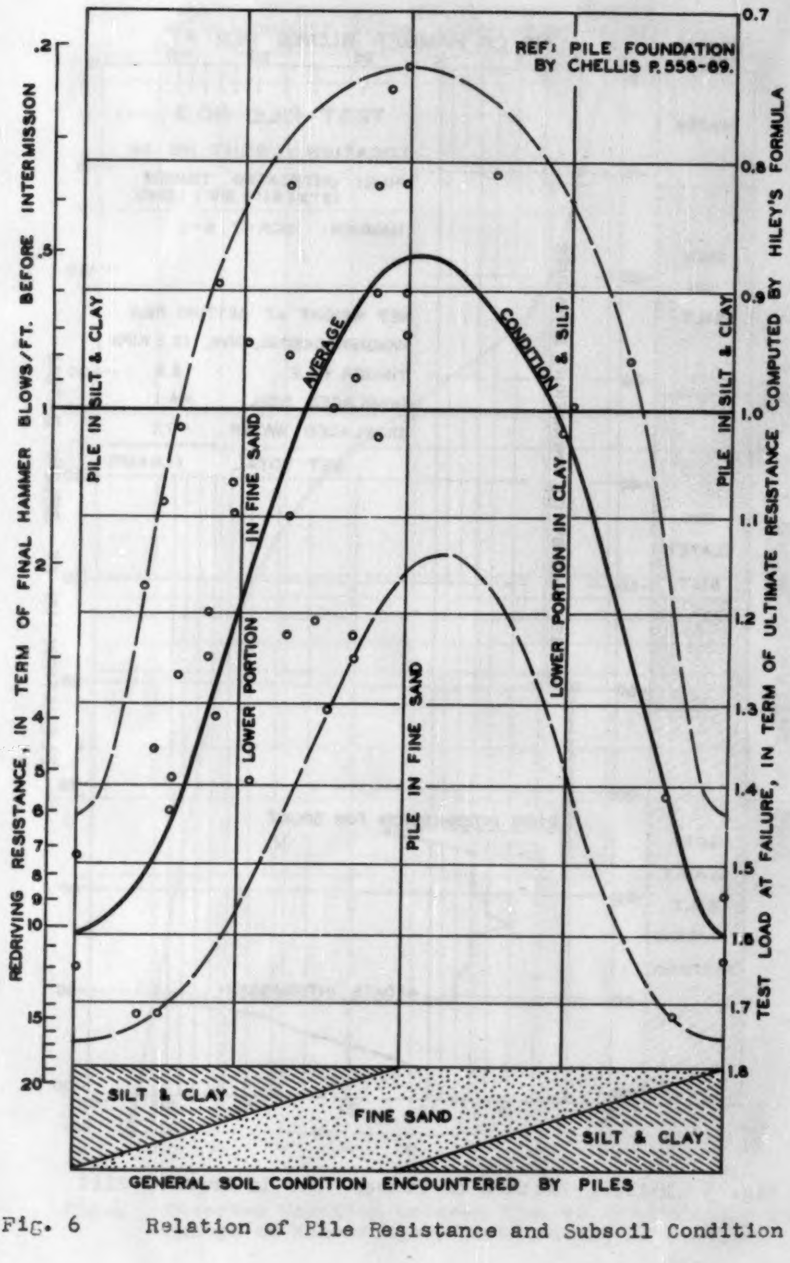

Fig. 6 Relation of Pile Resistance and Subsoil Condition

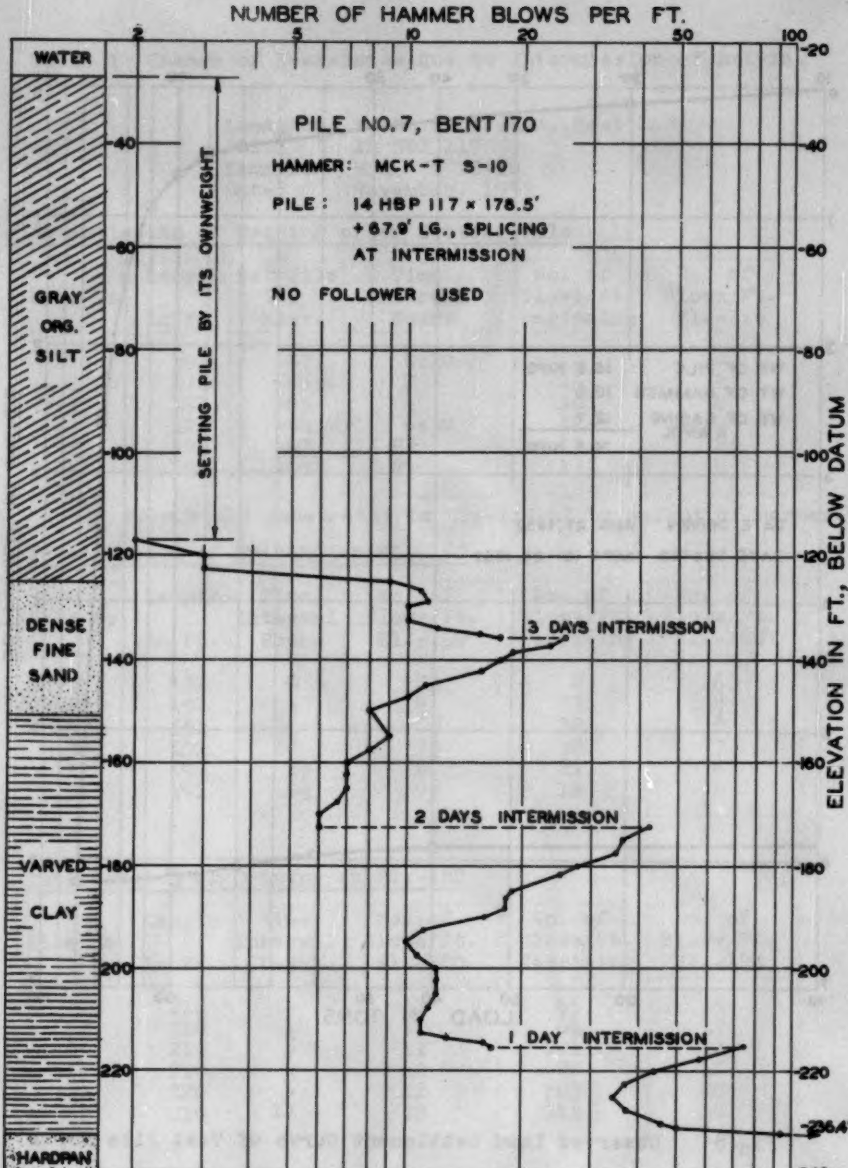

Fig. 7 Change of Driving Resistance due to Intermission for Splice

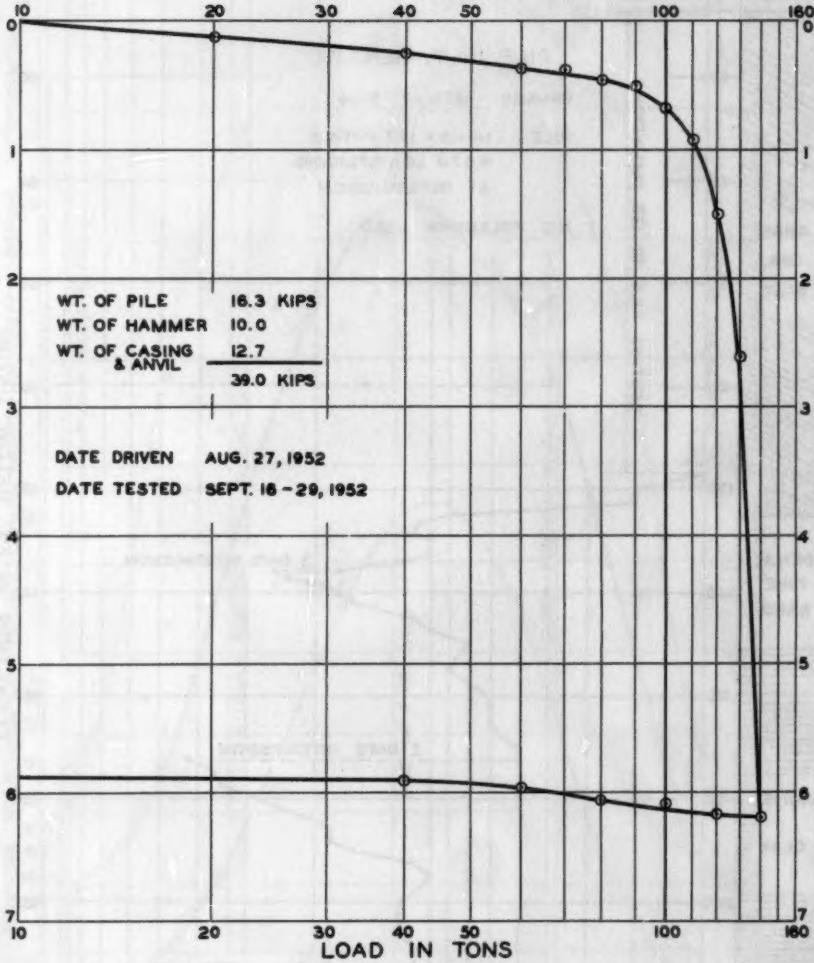

Fig.8 Observed Load Settlement Curve of Test Pile No. 1

Table I Change of Resistance due to Intermission of Driving

Location: North Cofferdam, Bent 168
Pile: 14 HBP 117
Hammer: McK - T S-10
Date: November, 1953

Intermission of Driving after Setting Pile

Pile No.	Length in Ft	Set Pile Tip Elev.	Time Interval Hours	No. of Blows/Ft. Beginning	No. of Blows/Ft. El.-110
19	191	-97	0:20	-	-
20	191	-95	2	15	2
37	191	-95	4	10	2
38	191	-94	6:30	12	2
22	191	-93	23	17	2½
51	191	-97	26	13	4

Pile No.19 penetrated to El.-119.4' by weight of hammer.

Intermission of Driving at El.-150

Pile No.	Length in Ft.	Time Interval Hours	No. of Blows/Ft. El.-150	No. of Blows/Ft. Redriving	No. of Blows/Ft. El.-160
19	191	0	7½	8	6
20	191	44	9	73	8
37	191	46	8½	72	8
38	191	18	7½	38	7
22	191	2½	8	21	6
51	191	0½	9½	16	6

Intermission of Driving at El.-180

Pile No.	Length in Ft.	Time Interval Days	No. of Blows/Ft. El.-180	No. of Blows/Ft. Redriving	No. of Blows/Ft. El.-190
26	218	1	12	65	15
42	218	1	11	64	17
25	219	5	11	101	17
39	219	4	10	80	17
40	220	5	12	103	20
11	219	11	12	112	18

Table II Comparision of Redriving Resistance of Timber & Steel H-Piles

12" Dia. Timber Piles in Organic Silt					14" Steel H-Piles in Organic Silt				
Embedded Length in Ft.	Blows/Ft at Inter mission	Starting Blows/Ft Redrive	Rate of Increase	Days of Inter-mission	Embedded Length in Ft.	Blows/Ft at Inter mission	Starting Blows/Ft Redrive	Rate of Increase	Days of Inter-mission
68	3	66	22.0	75	134	6	49	8.2	64
79	4	150	37.5	74	100	12	78/0.9	7.2	47
73	3	30/3"	40.0	74	137	8	48	6.0	19
69	3	39	13.0	75	129	9	126	14.0	20
60	2	44	22.0	76	136	4	46	11.5	18
61	2	31	15.5	73	135	4	65	21.2	18
60	2	38	19.0	73	126	8	90	11.2	20
64	3	36	12.0	51	124	7	74	10.6	21
58	2	31	15.5	51	133	7	91	13.0	20
60	1½	35	28.0	51	135	6	67	11.2	17
70	5	61	12.2	50	130	5	71	14.2	17
53	3	26	8.7	50	138	4	42	10.5	17
74	6	78	13.0	48	135	7	72	10.3	19
60	2	32	16.0	47	135	6	84	14.0	19
58	2	47	23.5	47	135	8	98	12.3	19
65	4	102	25.5	48	133	11	68	6.2	17
71	4	55	13.8	45	134	8	80	10.0	19
67	2	15/3"	30.0	45	133	7	54	7.7	18
		Average	20.4	59			Average	11.5	19

Table III Analysis of Driving Resistance due to Intermission as per Fig. 7

Notation: P = Point resistance of pile tip in dense sand
p = Point resistance of pile tip in varved clay
S = Skin friction on pile surface
a = Rate of change in point resistance at redriving
b = Rate of change in skin friction at redriving

Location of Pile Tip	Number of Blows/Ft.	Penetration "s" in Inch	Elastic Deformation "c" in Inch	Blows/Ft. Adjusted by $s/(s+\frac{1}{2}c)$	Relative Resistance
In Silt, El.-120'	3	4.00	0.02	3.0	p + S = 0.19
In Sand, El.-136'	18	0.67	0.20	15.7	P + S = 1.00
Redriving in Sand	25	0.48	0.27	19.5	aP + bS = 1.24
In Clay, El.-172'	6	2.00	0.05	5.9	p + S = 0.19
Redriving in Clay	42	0.28	0.26	28.7	ap + bS = 0.93

Analysis of Pile Resistance

S	p	P	aP	bS	$\frac{S}{p+S}$	$\frac{bS}{ap+bS}$	$\frac{S}{P+S}$	$\frac{bS}{aP+bS}$	b
					%	%	%	%	
.00	.19	1.00	.38	.86	0	92	0	69	
.02	.17	.98	.38	.87	11	93	2	70	43.2
.04	.15	.96	.37	.87	21	94	4	70	21.8
.06	.13	.94	.36	.88	32	95	6	71	14.7
.08	.11	.92	.35	.89	42	96	8	72	11.1
.10	.09	.90	.34	.90	53	96	10	72	9.0
.12	.07	.88	.34	.90	63	97	12	73	7.5
.14	.05	.86	.33	.91	74	98	14	74	6.5
.16	.03	.84	.32	.92	84	99	16	74	5.7
.18	.01	.82	.31	.93	95	100	18	75	5.1

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A BRIEF NOTE ON COMPRESSION INDEX OF SOIL

Yoshichika Nishida[1]
(Proc. Paper 1027)

SUMMARY

This paper reports a new relationship between the compression index and the void ratio of soils after a theoretical consideration from some simple assumptions.

The relationship given by the author agrees well with experimental results of verification and gives a new approximate method to estimate the compression index of soils as a linear function of the void ratio.

INTRODUCTION

For settlement analyses of the foundation it is an important thing to know the compression index of the soil, which is obtainable in the consolidation test. However there are some approximate estimations of the compression index, without doing consolidation tests in practice, according to other soil characters.

A. W. Skempton and others gave a relationship between the compression index and the liquid limit,(1) but some soils did not obey to the relationship(2) since coefficients would have to be determined in each kind of soils. K. V. Helenelund plotted a curve of the compression index versus the natural water content.(3) L. Bendel found some relationship, by experiments, between the compression index and the cone penetration, the simple compression strength, the triaxial strength, etc.(4) L. Marivoet used a formula of Buisman.(5) P. C. Rutledge expressed the compression index empirically as the function of the initial void ratio.(6) These relationships have been introduced only by plotting test results and have no theoretical base.

It seems to be more rational that the compression index will be expressed by the function of the void ratio where it is measured on the virgin curve of void ratio versus logarithm of pressure, because the index, which is obtainable from the tangent to the curve, is not constant at each point on the curve. Accordingly the author did the following calculation.

Note: Discussion open until December 1, 1956. Paper 1027 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 82, SM 3, July, 1956.

1. Instructor of Civ. Eng., Univ. of Kanazawa, Kanazawa-shi, Japan; formerly, Fellowship Eng., Instituto de Pesquisas Tecnologicas, Univ. of San Paulo, San Paulo, Brasil.

Compression of Soils

When a soil mass is compressed in all side directions by an external pressure, it deforms and the strain seems to be composed of the sum of the elastic deformation of soil grains, including the combining material between each grain, themselves and the decrease of voids between these grains. The former one deforms at once proportionally to the applied pressure, but the later one needs to take some time because it is the result of movements of grains along directions of minor resistance, accompaning the water drainage in some cases. When a pressure is applied to a soil mass which had been loaded once formerly, the deformation is somewhat smaller than the one elastically expected from the proportion between the pressure and the deformation in the former case because the voids had decreased by the pre-compression. The author assumes that the reduction from the value expected by Hooke's law is proportional to the quantity of deformation formerly received and to the time length during receiving the deformation.(7) Accordingly when a soil mass has been receiving a strain $\epsilon(\tau)$ during $\Delta\tau$ from time τ, the strain of the soil mass ϵ(t) at time t corresponding to an applied pressure p(t) will be expressed as follows:

$$\varepsilon(t) = \frac{1}{E} p(t) - B \cdot f(t-\tau) \cdot \varepsilon(\tau) \cdot \Delta\tau \tag{1}$$

where E is the coefficient corresponding to the elastic modulus which includes both the elastic deformation of and the void decrease between soil grains, and B is a characteristic constant for each kind of soils, and f(t - τ) is the time-function which shows the influence of the previously received deformation to the present state. If a pressure is applied continuously from t = 0, then

$$\varepsilon(t) = \frac{1}{E} p(t) - B \int_0^t f(t-\tau) \cdot \varepsilon(\tau) \cdot d\tau \tag{2}$$

When it is assumed that

$$f(t-\tau) = e^{-B(t-\tau)} \tag{3}$$

because the previous deformation of the infinitive time before (t - $\tau \rightarrow \infty$) will have no influence upon the present while the deformation of a moment before (t = τ) will give the full effect to the present, then the following equation is obtainable from eqs. (2) and (3) after differentiating.(8)

$$\frac{d\varepsilon}{dt} + 2B\varepsilon = \frac{1}{E}\left(\frac{dp}{dt} + Bp\right) \tag{4}$$

This is a fundamental relation between the stress and the strain of soils, introduced by the author, and it becomes Maxwell's equation if the second term of the left side is eliminated and also becomes similar to Voigt's equation if the first term of the right side is eliminated.

Stress-Strain of Soil in Statics

When a certain pressure of constant is applied on a soil mass the soil mass begins to deform, and after a certain time will have elapsed the deformation ceases to continue and the stress-strain relation becomes the one in state of static equilibrium. At this moment the deformation is recorded as the one corresponding to the applied pressure. Accordingly if the pressure increment of constant Δp is applied the increment of deformation $\Delta\epsilon$ corresponding to Δp is given by integration of eq. (4), after $d\,\Delta p/dt = 0$ and the initial condition of $\Delta\epsilon = 0$ for $t = 0$, as follows:

$$\Delta\varepsilon = \frac{1}{2}\frac{\Delta p}{E}\left(1 - e^{-2Bt}\right) \tag{5}$$

In a certain time, by putting $t \to \infty$ in eq. (5), the final deformation $\Delta\epsilon_{\infty}$ due to Δp is as follows:

$$\Delta\varepsilon_{\infty} = \Delta p / 2E \tag{6}$$

The Fig. 1 represents eq. (6) in simple. And it also gives an explaination to the fact that the ratio between the expansibility and the compressibility of soils does not exceed over[9] the value of 0.5 at maximum even if the soil grains rebound fully in the expansion process. If the pressure increment is applied continiously from $p = 0$, the following relation is obtained.

$$p = c\varepsilon^2 \tag{7}$$

where c is a constant, p is the pressure(stress) and ϵ is the deformation (strain).

Compression Index of Soil

It is able to introduce the following relation from eq. (7).

$$\frac{dp}{d\varepsilon} = 2c\varepsilon \quad \therefore \quad \frac{d\varepsilon}{dp} = \frac{1}{2c}\frac{1}{\varepsilon} = \frac{1}{2}\frac{\varepsilon}{p} \tag{8}$$

As the volume change of soil mass under an applied pressure seems to depend on the decrease of voids, eq. (8) gives as follows:

$$\frac{d}{dp}\frac{e_o - e}{e_o + 1} \doteqdot \frac{1}{2}\frac{1}{p}\frac{e_o - e}{e_o + 1} \tag{9}$$

where e_o is the void ratio before the pressure is applied, and e is the void ratio where the compression index is measured on the virgin curve of void-ratio versus log.-pressure. Eq. (9) will be

$$-\frac{de}{dp} = \frac{1}{2}\frac{e_o - e}{p} \quad \therefore \quad P\frac{de}{dp} = 0.5\,(e - e_o) \tag{10}$$

As the curve of void-ratio versus log.-pressure is, in general, plotted on the ordinary semi-logarithmic scale, the compression index C_c which is obtainable from the curve will be

$$C_c = \left| \Delta e \,/\, \Delta \log_{10} p \right| = 2.3 \left| p \, {de}/{dp} \right| \tag{11}$$

From eq. (10) and eq. (11) it will be

$$C_c = \left| 1.15 \, (e - e_o) \right| \tag{12}$$

The compression index will be equal to zero after having reached to the state of the closest packing, if soil grains are assumed to be incompressible. When soil grains are assumed to be uniform rigid sphere it is determined that e_o = 0.35 according to putting $C_c = 0$ for $e = 0.35$. Therefore,

$$C_c = 1.15 \, (e - 0.35) \tag{13}$$

This eq. (13) is the fundamental relation introduced by the author on the compression index and the void ratio. When the soil is compressed and soil grains of uniform rigid sphere are assumed to arrange as Fig. 2-b at its closest packing, according to $C_c = 0$ for $e = 0.91$ the relation becomes as follows:

$$C_c = 1.15 \, (e - 0.91) \tag{14}$$

If soil grains of incompressible but deformable are assumed, (See Fig. 2-c), according to $C_c = 0$ for $e = 0$ the relation becomes as follows:

$$C_c = 1.15 \, e \tag{15}$$

Accordingly all points plotted by C_c versus e seem surely to distribute along the line of eq. (13) between the line of eq. (14) and the line of eq. (15). It must be noted that the C_c should be, of course, measured at e which is far from the influence of pre-consolidation on the virgin curve of void-ratio versus log.-pressure.

Compression Index vs. Initial Void Ratio and Natural Water Content

The relation between the compressive pressure and the void ratio of soils is expressed as follows:

$$e = e_o - C_c \, (\log_{10} p = \log_{10} p_o) \tag{16}$$

where p_o is the pressure corresponding to the initial void ratio e_o and p is the pressure corresponding to the void ratio e where the compression index is obtainable. After putting eq. (16) into eq. (13) it follows that

$$C_c = \frac{1.15 \, (e_o = 0.35)}{1 + 1.15 \log_{10} (p/p_o)} \doteqdot 0.54 \, (e_o = 0.35) \tag{17}$$

because $\log_{10}(p/p_o)$ is approximately assumed to be usually equal to the unit when C_c is asked. If it is assumed that the voids of soil mass is saturated with water and that the density of soil grains is about 2.60, the eq. (17) becomes as follows:

$$C_c = 0.54\,(2.6\,W_o - 0.35) \tag{18}$$

where W_o is the initial water content and it is generally same as the natural water content.

Comparison with Experimental Results

Fig. 3 shows the comparison between such a new formula by the author as eq. (13) and experimental results, where the compression index versus the void ratio is plotted. Fifteen kinds of soils in four parts of Brasil were used in tests. The test results shows that the author's idea is appropriate to practice.

Fig. 4 shows the comparison between eq. (13) and test results done by P. C. Rutledge.[10] The compression index was obtained by measuring on many curves of his consolidation tests for some clays.

Fig. 5 shows the comparison between eq. (13) and test results done by P. C. Rutledge[11] on Mexico City Clay. Good conformity of the author's theory with the practical test is proved.

Fig. 6 shows the comparison between eq. (17) and test results done by P. C. Rutledge[12] on the compression index versus the initial void ratio. Some discord between the theory and the practice is found at the higher initial void ratio. The gap seems to be brought by putting that $\log_{10}(p/p_o)$ is equal to unity in eq. (17). At higher initial void ratio $\log_{10}(p/p_o)$ must be smaller than unity, and if it is so the gap becomes smaller.

Fig. 7 shows the comparison between eq. (17) and test results done by G. F. Sowers on the compression index and the initial void ratio.[13] Good conformity of the theory and the practice is proved.

Fig. 8 shows the comparison between eq. (18) and test results[14] done by K. V. Helenelund on the compression index and the natural water content for seven kinds of soils from clay to silt. Good conformity is proved between the theory and the practice.

Fig. 9 shows the comparison between eq. (18) and test results done by P. C. Rutledge on the compression index versus the natural water[15] content for four kinds of clays. The theory agrees well with the practice.

CONCLUSION

According to many test results in practice the variation of initial void ratio in the natural ground is approximately a linear function of the liquid limit of the soil. The liquid limit (and plastic index) is proportional to the clay content in soils, and the compression index is proportional to the clay content, too. And so eq. (13) seems to have the same meaning as Skempton's formula. However the author's formula eq. (13), introduced in theoretical, agrees[16] well with test results for any kind of soil, and so it will give a new approximate method to look for the compression index of soils, basing on the more rational point of view than others.

ACKNOWLEDGMENT

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14. K. V. Helenelund: ditto - (3) p. 30.
15. P. C. Rutledge: ditto - (6) p. 143.
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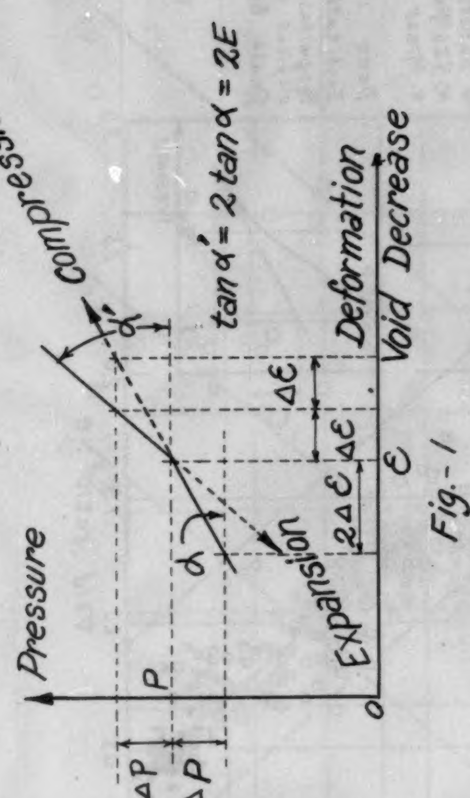
Pressure
Compression
Expansion
Deformation
Void Decrease
$\tan\alpha' = 2\tan\alpha = 2E$
ΔP
ΔP
P
ε
$\Delta\varepsilon$
$\Delta\varepsilon$
$2\Delta\varepsilon$
α
α'
0

Fig.-1

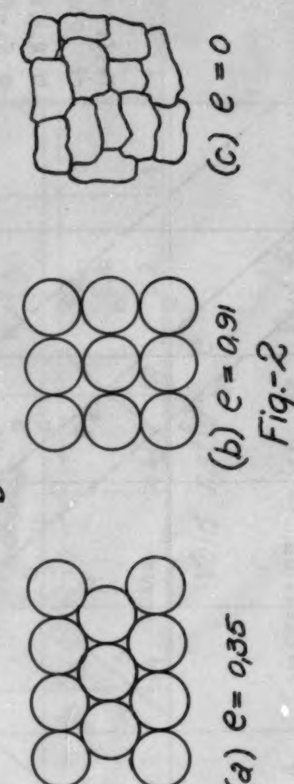
(a) e = 0,35
(b) e = 0,91
(c) e = 0

Fig.-2

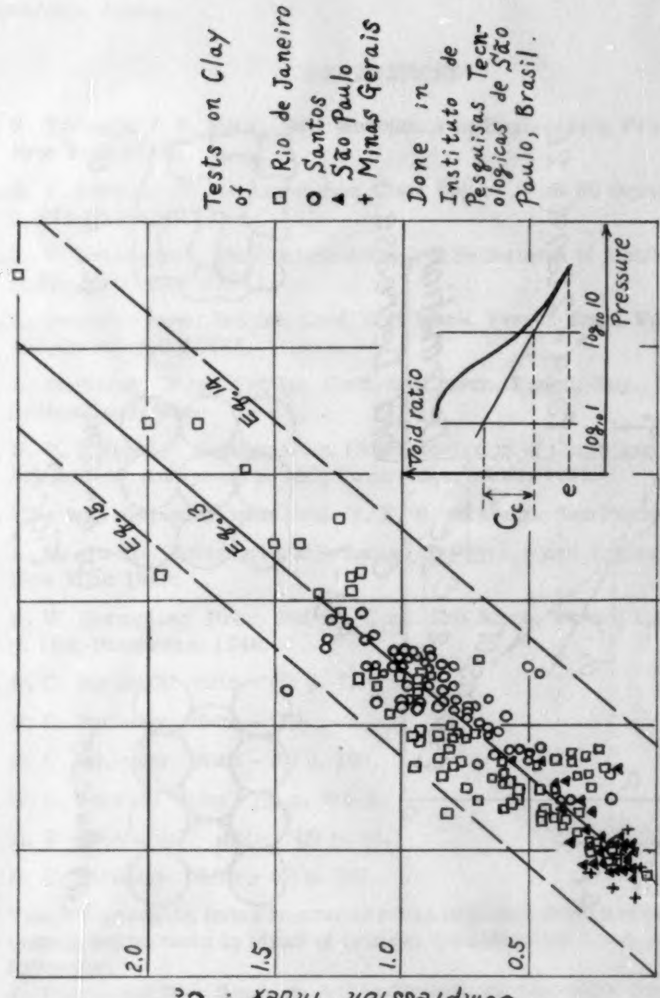

Fig. 3 Author's Line v.s. Test Results on Compression Index v.s. Void Ratio

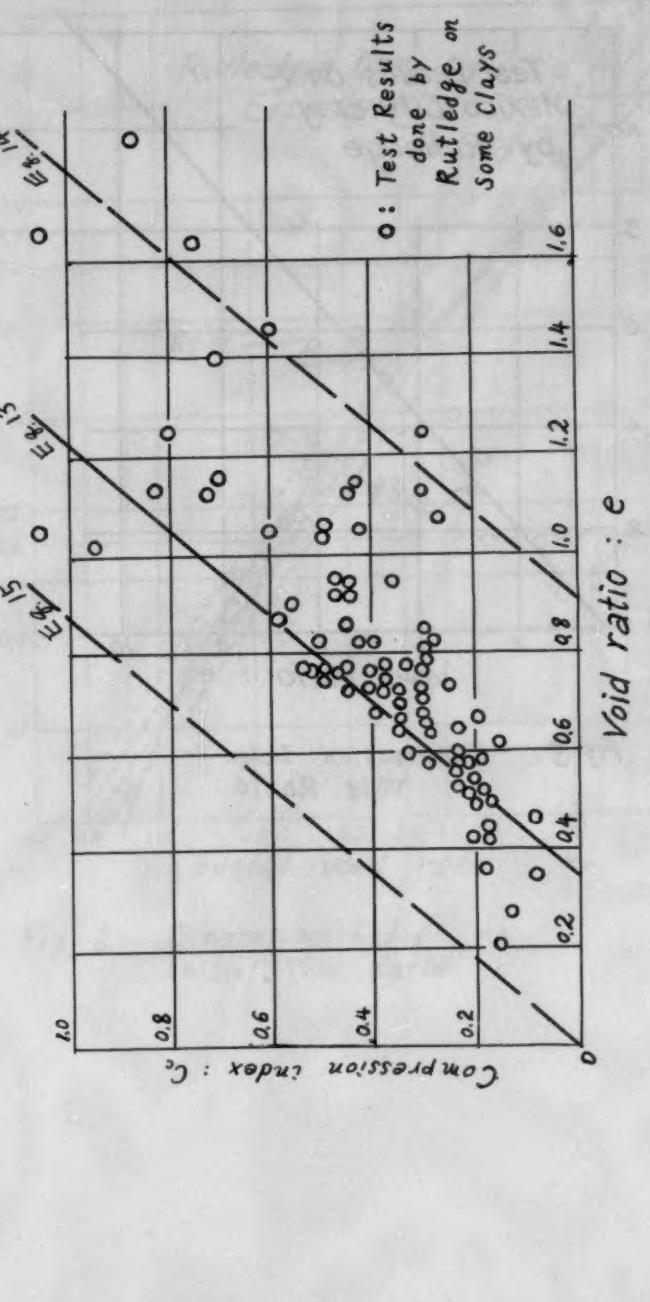

Fig.-4 Compression Index v.s. Void Ratio

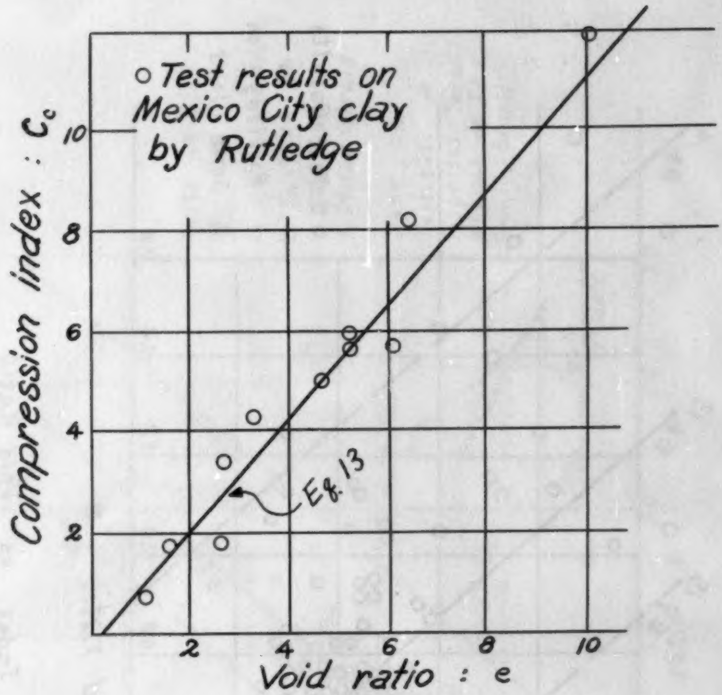

Fig.-5 Compression Index v.s. Void Ratio

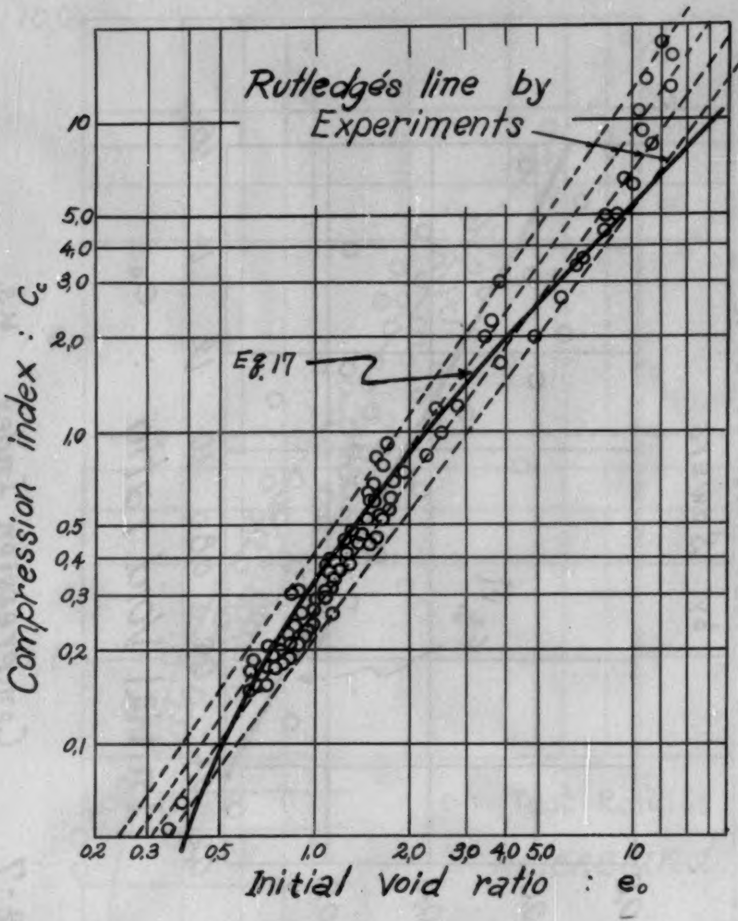

Fig.- 6 Compression Index v.s. Initial Void Ratio

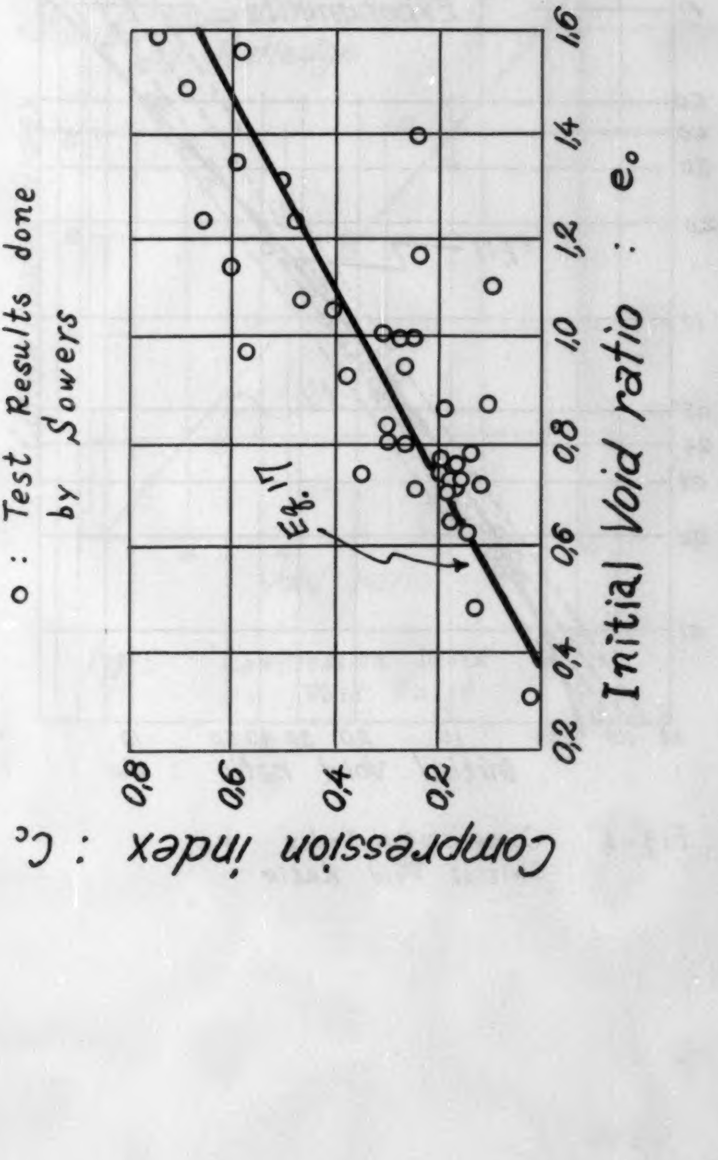

Fig.-7 Compression Index v.s. Void Ratio

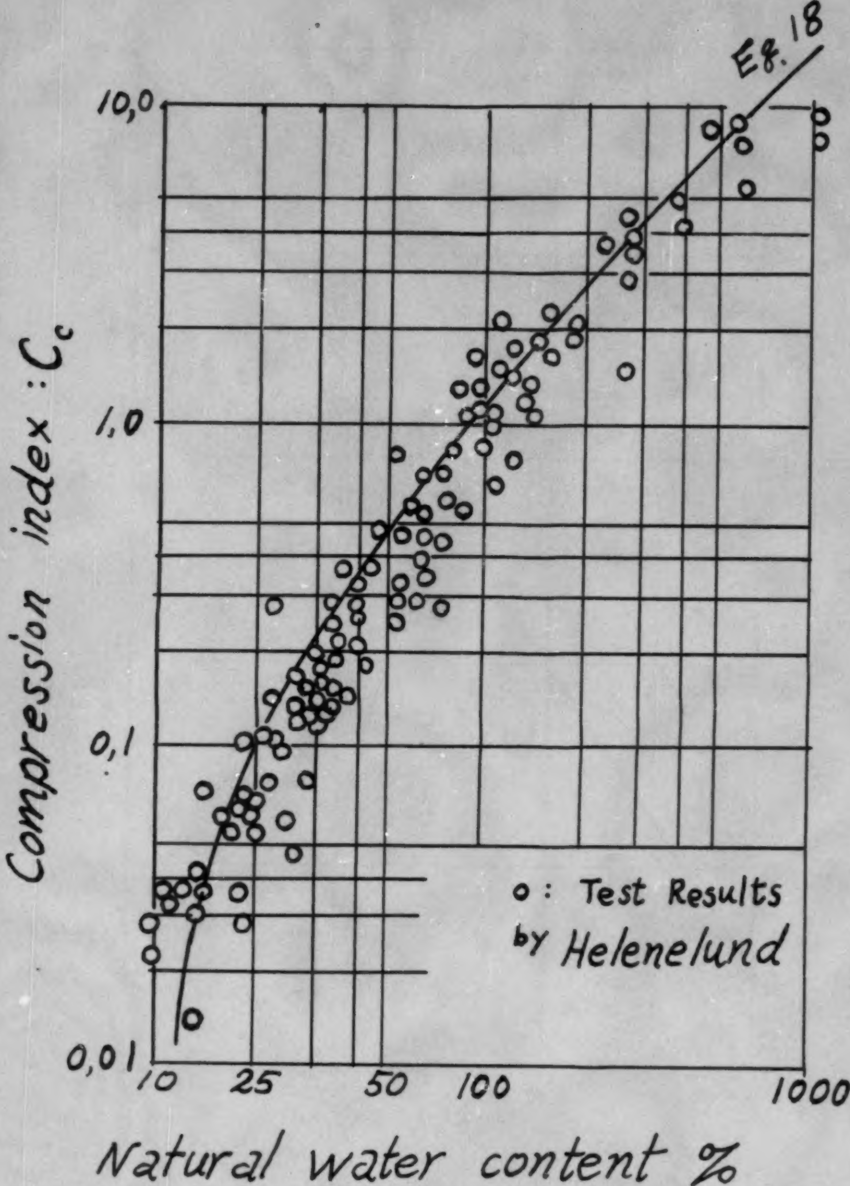

Fig.- 8 Compression Index v.s. Natural Water Content

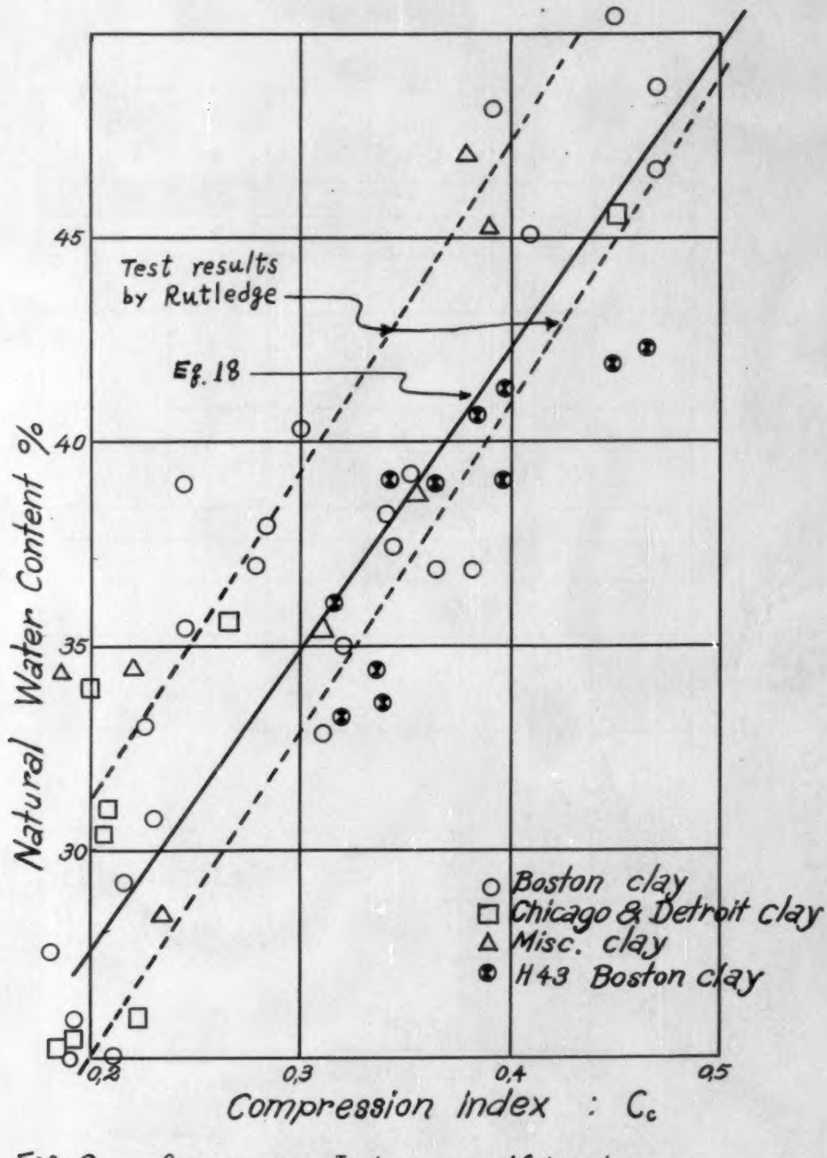

Fig.-9 Compression Index v.s Natural Water Content

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1028)

(Over)

Discussion of "FIELD VANE SHEAR TESTS OF SENSITIVE COHESIVE SOILS"

by Hamilton Gray
(Proc. Paper 755)

HAMILTON GRAY,[1] M. ASCE.—In commenting upon the welcomed and thoughtful discussions of this paper the writer first reiterates a statement made on page 755-7.

> "It appears very doubtful that the true strength of sensitive soils can ever be directly measured by small-scale mechanical means other than that represented by a plate bearing test, since no claim can be made that the insertion of a vane does not to some extent disturb and weaken the adjacent soil. The results of vane tests made at the bottom of drill holes appear to merely represent a closer approach to the true strength values than can be expected from results of tests performed upon even the best of so-called undisturbed samples."

This statement recognizes the fact that the insertion of any object, be it a vane or a sampling device, into the soil produces some displacement and distortion so that the strength determinations which follow such insertion must reflect, to some extent, the effect of soil disturbance. The amount of distortion and the effect thereof on measured strength, varies not only with the magnitude and configuration of the displacement but also with the particular soil i.e., certain soils, at appropriate consistency, when cut or indented, tend to undergo relatively large deformations remote from the area of applied stress whereas at softer or stiffer consistencies the separation or indentation causes but negligible deformation at remote points. The effect of distortion upon strength also seems to vary with soil consistency and the ratio of natural water content to liquid limit.

The insertion of a large vane in a sample tube is obviously undesirable, but the use of a miniature vane in the field introduces different complications. Consequently, a comparison between field and laboratory vane tests wherein one and the same vane is used does not appear practicable except at very shallow depths.

It is clear that the presence of granular varves introduces a complicating factor. Thickness of the individual varves is probably the most significant item, for friction on the circular periphery of a very thin varve can hardly be important. Preliminary investigations have demonstrated that it is possible to rotate a vane in loose cohesionless soil. Sampling of such soils in a manner which will not greatly alter their density requires very elaborate procedures and is normally impracticable. Hence some type of testing in situ is desirable, but whether vane testing of loose cohesionless deposits can yield valuable information remains to be seen. However, it is suggested that when a soil containing relatively thick varves of cohesionless material is

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sampled, the density of the granular material may be altered so that when samples are tested in triaxial compression unduly high results are obtained. Furthermore, when a highly sensitive, purely cohesive, soil is sampled and then subjected to consolidating pressures equal to the existing overburden or effective field pressure, the unavoidable sampling disturbance coupled with reconsolidating pressures will produce a void ratio which is smaller than the value in the original ground. Shearing the material at this reduced void ratio will result in a greater observed strength than that possessed by the natural deposit.

There is no comparison in the time required and cost involved in performing the field vane and triaxial tests and hence if vane tests are reliable they are economically important. The writer is just as reluctant to accept the thesis that sampling and handling sensitive materials do not affect laboratory test results as others are to abandon it. Both attitudes are subjective and one is not likely to change a subjective attitude unless it is challenged. The writer seriously doubts whether it can be proved that tests of samples of highly sensitive soils are more reliable than field tests and feels that up to the present time no semblance of such proof has been presented. Such proofs are not likely to result from a complacent attitude. Finally irrespective of the cause of the differences in field vane and laboratory test results, when such differences do occur it is likely that one value is more reliable than the other and consequently a choice must be made as to which value is the appropriate one to use. The writer has not been persuaded that direct use of or extrapolation from laboratory test results is always as reliable as measurement of mechanical properties in the natural deposit in the field, and hence is inclined to favor the field tests.

In conclusion, the writer expresses his thanks to the various discussers and adds the hope that others as well as they, will undertake to answer some of the uncertainties which are inherent in this aspect of soil mechanics.

Discussion of
"THE ACTION OF SOFT CLAY ALONG FRICTION PILES"

by H. B. Seed and L. C. Reese
(Proc. Paper 842)

LAWRENCE A. DUBOSE,[1] A.M. ASCE.—The interesting paper by Messrs. Seed and Reese can be used to illustrate an important deficiency in the field of soil mechanics and foundation engineering. This deficiency is the lack of rational and standard pile load test procedures which associate the tolerable amount of pile deformation during the load testing program with stress and strain properties of the foundation medium. The influence of time should also be considered in designing the load test procedures.

Despite the fact that the authors recognized their load test procedure differed "from that recommended by most authorities in that the time between load increments was quite short," the influence of the short time interval on load-settlement relationships was not evaluated. Since an attempt was made by the authors to correlate load test results with computed values based on soil strength values, the correctness of the load test procedure used becomes of utmost importance.

The necessity of using a quick or short term load test procedure is recognized in evaluating the pile load capacity four hours after the pile had been driven. However, the phenomenon that pile driving remolds soft clay deposits and causes an initial low supporting capacity for the pile has been observed and reported numerous times. Therefore, the load supporting capacity of the pile immediately after driving appears to be primarily of academic interest. Of more practical value is the load carrying capacity of the pile at some elapsed time interval related to actual construction loadings. Load tests conducted for this condition would permit the time factor to be considered. While it is recognized that the method of placing the pile is important, it is felt that the time factor deserves consideration in performing load tests, whether the pile has been driven or drilled and cast-in-place.

The writer would like to present the results of load tests conducted on drilled and cast-in-place concrete piles for which the time factor was considered. Figure 1 shows load versus settlement curves for a long term and a short term test conducted on a drilled and cast-in-place concrete pile. The pile diameter was fourteen inches, the effective length was nineteen feet and the total depth of the pile was twenty-one feet. The long term test was continuous for a 194 hour period with four dial gages being read to the nearest 0.001 inches at 15 minute intervals. (The first four tests reported by the authors were conducted within about 168 hours after the pile had been driven. The actual testing time was probably only a small fraction of this time period.) For the long term tests, the pile load was not increased until the settlement within a four hour period had not exceeded 0.010 inches. This was intended to represent a static equilibrium condition. The short term test was conducted by increasing the pile load in 10 kip increments at five minute

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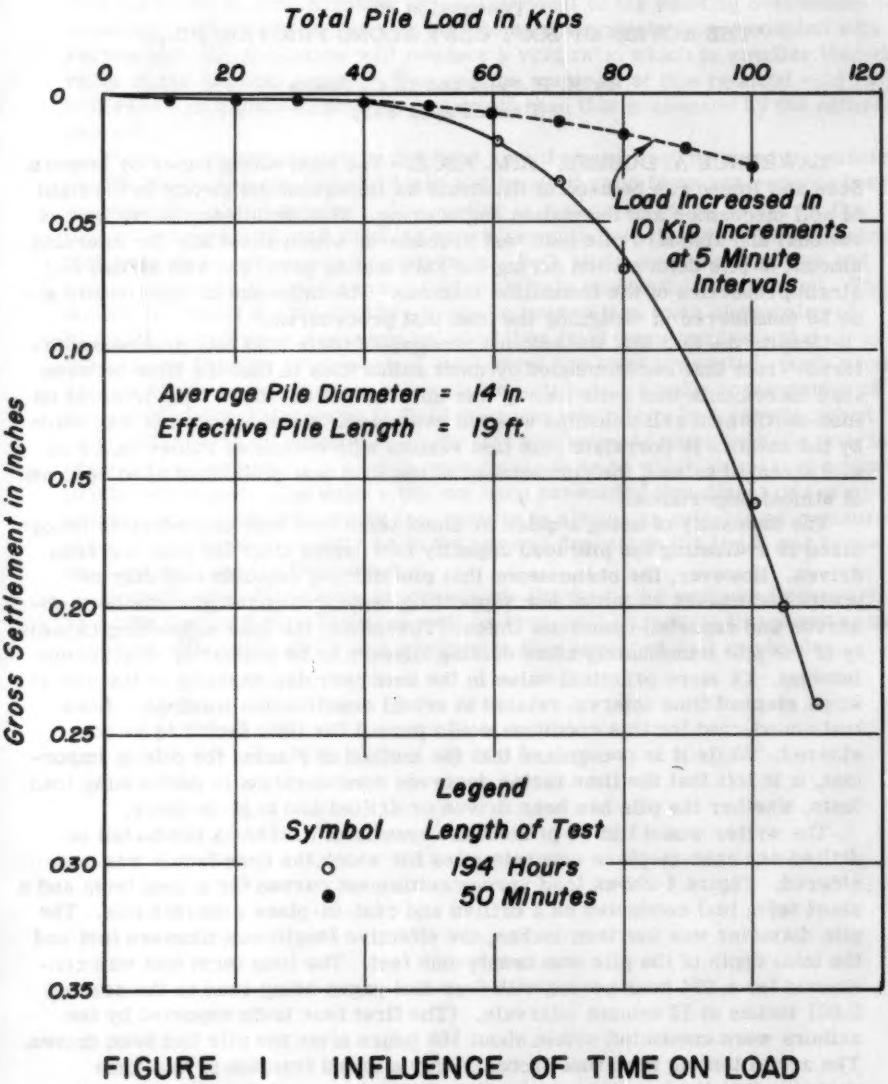

FIGURE 1 INFLUENCE OF TIME ON LOAD VERSUS SETTLEMENT RELATIONSHIPS

intervals. The total time for this test was 50 minutes. One may note an increasing divergence of the two curves which begins at about 50 kips. Under a total load of 100 kips the pile had settled about 0.16 inches during the long term test and only about 0.03 inches for the short term test. Similar relationships were established for other piles subjected to both long term and short term tests. Tests were also conducted for which the pile loads were increased at five, fifteen and thirty minute intervals. There was little difference in load versus settlement relationships for load tests conducted using these three time increments for increasing the pile load. However, the settlement occurring during the short term load tests, as in the case of data shown in Figure 1, was much less than the settlement which occurred during the long term load test.

The soil at the test site for these piles is a stiff, desiccated clay with unconfined compression strengths increasing from about 0.7 tons per square foot at a depth of 2 feet to over 2 tons per square foot at a depth of 21 feet. Liquid limit values ranged from 35 to 80 per cent and plastic limit values were between 18 and 30 per cent. The in situ water content was approximately equal to the plastic limit. The soils at the driven pile site (reported by the authors) had lower strengths and higher water contents and should have been much more compressible than the clay at College Station. It appears that time would have been of even greater importance in conducting the load tests for the driven piles at the San Francisco Bay site.

Another important factor noted in conducting the tests on concrete piles was the increase in load capacity with successive tests. This is illustrated by the load versus settlement curves in Figure 2. While the retests were conducted at approximately 6 and 12 month time intervals after the initial test, the same general relationship was observed for retests conducted at shorter elapsed time periods. For the data shown in Figure 2, the pile load capacity increased from about 45 kips for the July, 1954 test to about 54 kips for the December, 1954 test and to over 60 kips for the July, 1955 test. The increase in pile capacity was associated with the retesting of the pile and not to an increase in soil strength resulting from in situ water content changes.

The writer feels that both of the above discussed factors must be evaluated before the validity of a load test procedure can be established. Long term load tests are time consuming and expensive and many engineers would like to have a quick or short term procedure for test loading piles. However, the correctness of data should not be sacrificed for this reason. The engineer realizes that consolidation of a soft clay deposit under a loaded area will continue for weeks or even years. Therefore, it is not logical to assume that the correct relationship between load and settlement can be established by loading a pile for a few hours without first comparing long term and short term load test data.

YOSHICHIKA NISHIDA.[1]—The authors have made many facts clear quantitatively such as remoulding effect and time effect after pile driving on soil properties, although those effects were discussed only qualitatively until today. They carried out not only the load test on piles but also showed the load distribution along a pile; the later data, though a few people already did such

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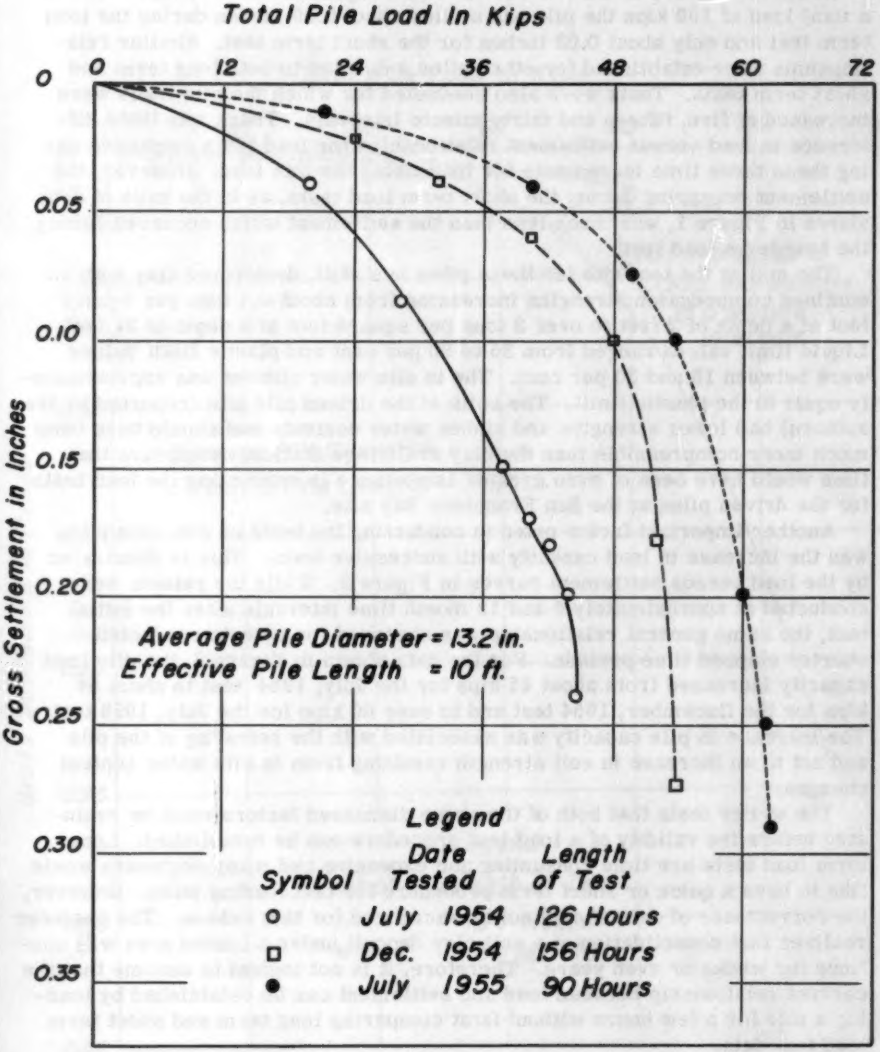

FIGURE 2 INFLUENCE OF SUCCESSIVE LOAD TESTS ON LOAD VERSUS SETTLEMENT RELATIONSHIPS

tests on the load transfer from a pile to the surrounding soil, seems to be most useful for the study of pile problems. They also explained how to compute the bearing capacity and the load distribution of a pile on the basis of data measured in practice.

The writer wishes to pay his respects to their great contribution as above and to send a brief discussion.

The writer thinks that the deformation in surrounding soils produced by a loaded pile is zero and is not maximum at the ground surface because there is no shear resistance (shear stress, friction). The deformation will be larger at some point of mid-depth alongside of a pile because the friction stress (load transfer) along the pile shaft seems to be larger there as easily seen from the authors' test Fig. 6, and the writer also obtained the same results after a theoretical calculation.

The authors give a differential equation (3), however there seems to be no direct relationship between the Eq. (3) and their numerical example. The writer hopes to know what Eq. (3) contributes. It can not be solved unless the rigorous knowledge of β is given, which is variable with depth as a function of ϕ itself.

The writer thinks that the methods of elasticity are useful for this pile problem, although they are of limited assistance, if applied to the state just before failure of soils and if the soil is assumed to be elastic when compared with the pile being assumed to be rigid. The writer obtained and solved the following equation, under those conditions of boundary and of yields on soil, governing the soil deformation surrounding a pile.

$$\left(\frac{\partial^2}{\partial r^2}+\frac{1}{r}\frac{\partial}{\partial r}-\frac{1}{r^2}+\frac{\partial^2}{\partial z^2}\right)^2\frac{\partial w}{\partial r}=0$$

where w is the vertical deformation of surrounding soil.

Of course the shear strengths determined from the vane test are higher than the maximum values of the rate of load transfer measured on the test pile, due to the presence of shells. But at the same time one should take care of the fact that there must be some difference of shear strength between pile to soil and soil to soil if the failure occurs at the interface between pile and soil as the authors found, and that only part of the length of pile shaft develops the maximum resistance of soil and not the full length of the pile contributes.

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Anais da Assoc. Brasil. d. Mecanica dos Solos (Trans. of Brasilian Assoc. on Soil Mechanics), Vol. 3, p. 17, 1953

2. Yoshichika Nishida: Determinacao do atrito lateral de estacas de fundacao, cravada, no caso de solos coesivos (A computation of the skin friction of a foundation pile driven into the cohesive soil)
Anais Assoc. Brasil. Mec. Solos (Trans. Brasil. Assoc. Soil Mech.), Vol. 4, 1954 (in press).

JAMES D. PARSONS,[1] M. ASCE, and RALPH B. PECK,[2] M. ASCE.—The authors have added valuable data to the store of information concerning the behavior of soft clay into which piles have been driven. It is noted that the average water content decreased from the original value of 48.1% to 43.6% adjacent to the pile within one day after pile driving, and to 41.1% within one month after pile driving. Although the scattering of the values of water content is appreciable, the general decrease in the neighborhood of the pile appears well substantiated, particularly in view of the corresponding increase in unconfined compressive strength.

The data obtained by the authors indicate a substantial degree of remolding adjacent to the pile, in spite of the relatively small diameter of each pile and the small number of piles in the group. The scattering of the values of water content, however, leaves considerable room for interpretation regarding the position of point B, Fig. 12, which established quantitatively the degree of remolding.

The writers have made similar studies of the degree of remolding associated with the driving of displacement piles to end bearing through soft deposits of clay at a site in the Chicago area. The soil profile is shown in Fig. 1, together with values of the natural water content, liquid limit, plastic limit, and unconfined compressive strength of undisturbed samples taken in a 3-in. diameter thin-walled piston sampler under carefully supervised field conditions. Between about El. 0 and El. -15 the clay is quite homogeneous. The homogeneous layer is slightly overconsolidated except at its upper surface where it has been rather highly preconsolidated by desiccation.

Piles were used to support a large heavily loaded structure at the site. For the most part they were driven from the surface of an area excavated to El. +7 or to El. 0. The upper part of each pile consisted of a corrugated metal shell with 12-in. diameter. The shell was generally about 35 ft. long. The lower part of the pile, about 15 ft. in length, consisted of a closed-end 10-in. steel pipe pile. The entire pile was eventually filled with concrete.

In much of the area the pile clusters were spaced at 14 x 17 ft. and consisted of some 9 to 16 piles each, spaced on about 3-ft. centers. Thus the density and volumetric displacement of the piles were relatively great. Subsequent to pile driving, 2-in. Shelby tube samples were taken in the area of disturbance approximately 104 days after pile driving had been completed in the vicinity. At a time approximately 410 days after pile driving, two additional borings in the disturbed area were made, from which 3-in. piston samples were taken. Values of water content and unconfined compressive strength were obtained from both sets of borings. In addition, consolidation tests were performed on the samples taken from the 3-in. boring, for comparison with those from the undisturbed area where 3-in. piston samples were also recovered. The results of typical consolidation tests are shown in Fig. 2.

Figure 2 also contains the results of average values of the unconfined compressive strength for the undisturbed samples within the homogeneous soft clay portion of the deposit, for completely remolded samples at the original water content, and for the tests performed on samples taken 104 and 410 days after pile driving. The average strength and water content of the undisturbed

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material are represented by point A, whereas those of the remolded material at natural water content are indicated by C. Points representing the average strength and water content of samples from the areas of pile driving are indicated by the triangular symbols. They define a line parallel to that of the straight portion of the e log p curve for a typical sample from the disturbed area. This is in agreement with the concept utilized by the authors to represent their results. The upward projection of the strength line to point B indicates the strength of the soil immediately after pile driving. The position of point B suggests that the soil had lost all but about 15 percent of its strength immediately after pile driving, or that it was almost completely remolded.

These results, together with those obtained by the authors, appear to indicate that relatively insensitive soft clay soils are likely to be substantially or almost completely remolded by pile driving.

It is hoped that the authors will indicate the location of the pore pressure gauges shown in Fig. 14. The large gradient in pore pressure and in strength to be expected in radial directions may account for some of the differences noted between the capacity of the pile and the progress of consolidation at the points where observations were made. It would also be of interest to see typical time-settlement curves for some of the load increments placed during the pile tests.

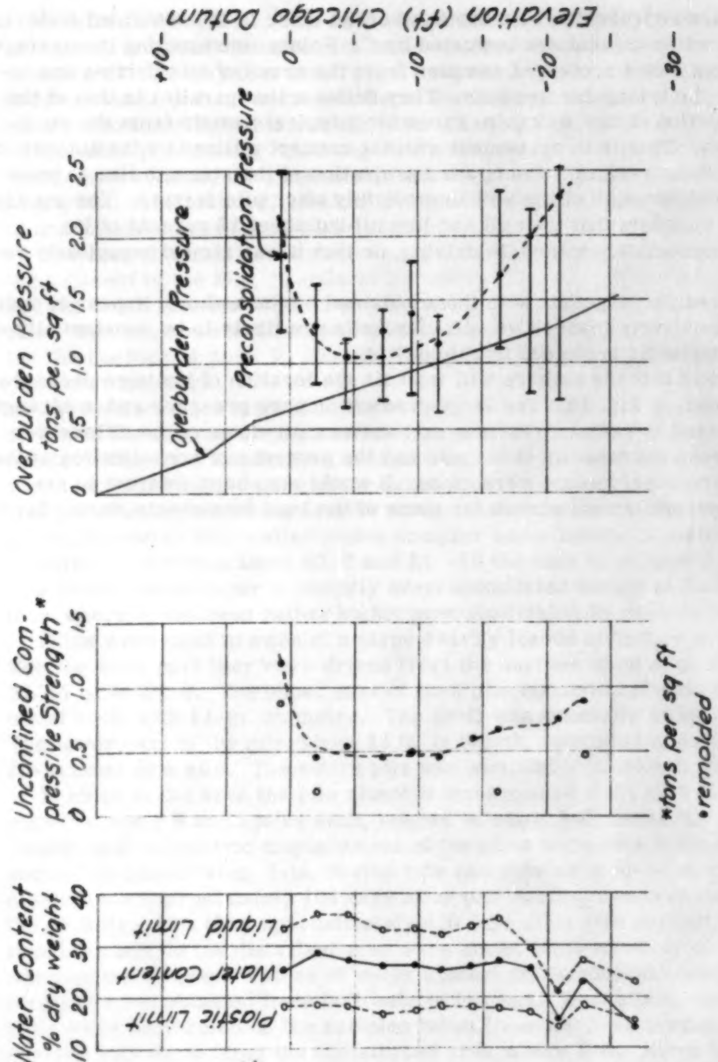
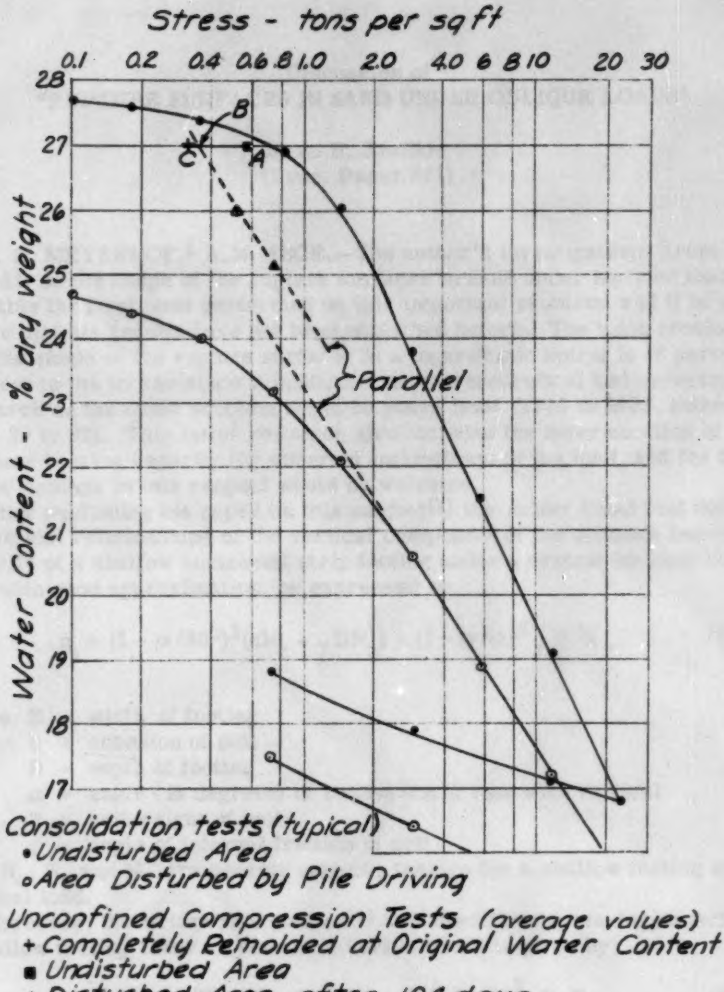
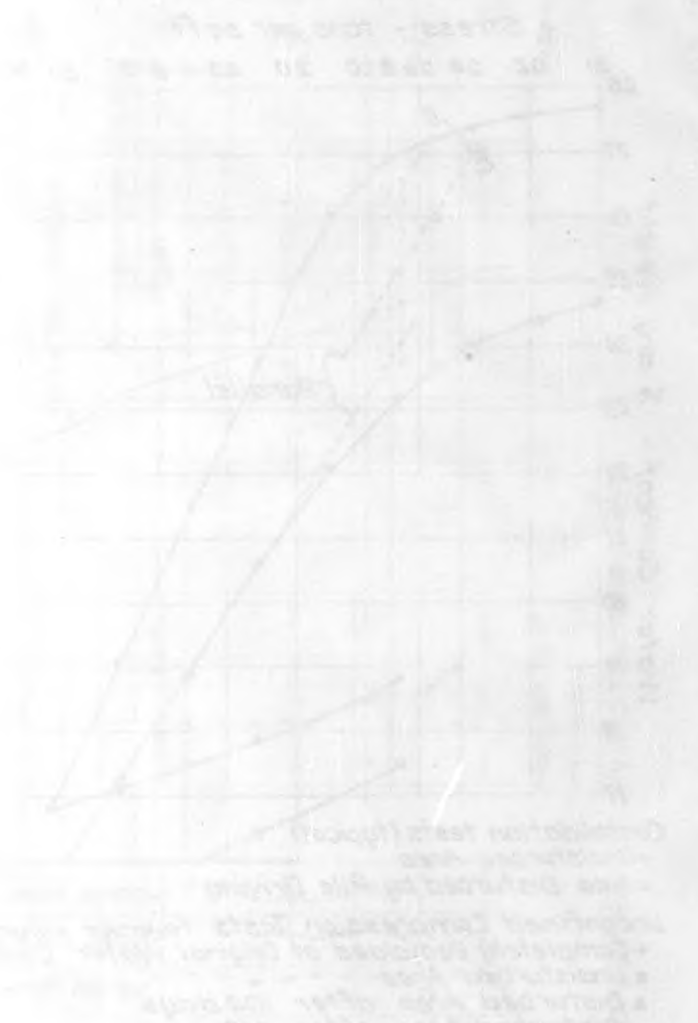


FIG. 1 PROPERTIES OF UNDISTURBED SOIL



STRESS AND STRENGTH vs WATER CONTENT RELATIONSHIPS
FIG. 2



Stress and Strength vs Water Content Relationships

Fig 2

Discussion of "RUPTURE SURFACES IN SAND UNDER OBLIQUE LOADS"

by A. R. Jumikis
(Proc. Paper 861)

G. G. MEYERHOF,[1] A.M. ASCE.—The author's investigations (from 1939 to 1942) on the shape of the rupture surfaces in sand under inclined loads are probably the first ones performed on this important problem, and it is unfortunate that his results have not been published before. The main conclusion that the shape of the rupture surfaces is a logarithmic spiral is of particular interest to the writer since it confirms his own theoretical and experimental research on the same problem some 10 years later (1948 to 1953, author's refs. 20 to 23). This latter research also included the determination of the ultimate bearing capacity for different inclinations of the load, and the author's findings in this respect would be welcome.

Since publishing his paper on this subject(1) the writer found that his theoretical relationships of the vertical component of the ultimate bearing capacity of a shallow horizontal strip footing under a central inclined load can, with good approximation, be expressed by

$$q_i = (1 - \alpha/90^\circ)^2(cN_c + \gamma DN_q) + (1 - \alpha/\phi)^2 \gamma \frac{B}{2} N_\gamma \qquad (1)$$

where B = width of footing
c = cohesion of soil
D = depth of footing
α = angle (in degrees) of inclination of load with vertical
γ = unit weight of soil
ϕ = angle of internal friction of soil

and N_c, N_q and N_γ are bearing capacity factors for a shallow footing with vertical load.

Moreover, the writer also showed(1) that the ultimate bearing capacity of a shallow footing under an eccentric vertical load is given by

$$q_e = (1 - 2e/B)\,(cN_c + \gamma DN_q) + (1 - 2e/B)^2 \gamma \frac{B}{2} N\gamma \qquad (2)$$

where e = eccentricity of load.

Hence combining Eqs. 1 and 2, the ultimate bearing capacity of a shallow horizontal strip footing under an eccentric inclined load may be expressed by

$$q_{ei} = (1 - 2e/B)\,(1 - \alpha/90^\circ)^2(cN_c + \gamma DN_q) + (1 - 2e/B)^2(1 - \alpha/\phi)^2 \gamma \frac{B}{2} N\gamma \qquad (3)$$

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The above relationships are supported by the test results to which the author refers (author's ref. 22), and comparison with his own investigations should prove of interest.

REFERENCE

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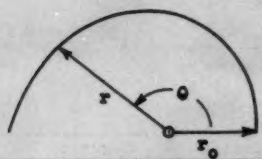
EDWARD S. BARBER,[1] A.M. ASCE.—Logarithmic spiral failure surfaces recommended in this paper were recently used by the writer for analyzing footing stability in a steep ash slope. The accompanying spiral tables are helpful in the application of spiral surfaces.

Table A gives spiral radii at various central angles for different friction angles. Table B gives the area of a spiral sector; the moment of cohesion along the spiral is 2c x the area of the corresponding sector. The moment of the sector, given in Table C in relation to a horizontal r_0, is required in considering the weight of displaced material.

1. Civ. Engr., Arlington, Va.

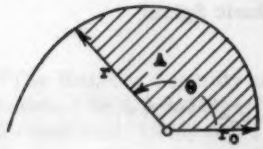
Table A - Radius of Logarithmic Spiral



$$r = r_o e^{\theta \tan \phi}$$

θ	Angle of internal friction, φ - degrees: 0	5	10	15	20	25	30	35	40	45
	Relative radius, r/r_o									
Degrees										
0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
5	1.00	1.01	1.02	1.02	1.03	1.04	1.05	1.06	1.08	1.09
10	1.00	1.02	1.03	1.05	1.07	1.08	1.11	1.13	1.16	1.19
15	1.00	1.02	1.05	1.07	1.10	1.13	1.16	1.20	1.23	1.30
20	1.00	1.03	1.06	1.10	1.14	1.19	1.22	1.28	1.34	1.42
25	1.00	1.04	1.08	1.12	1.17	1.22	1.29	1.36	1.44	1.55
30	1.00	1.05	1.10	1.15	1.21	1.28	1.35	1.44	1.55	1.69
35	1.00	1.06	1.11	1.18	1.25	1.33	1.42	1.53	1.67	1.84
40	1.00	1.06	1.13	1.21	1.29	1.38	1.50	1.63	1.80	2.01
45	1.00	1.07	1.15	1.24	1.33	1.44	1.57	1.73	1.93	2.19
50	1.00	1.08	1.17	1.26	1.37	1.50	1.65	1.84	2.08	2.39
55	1.00	1.09	1.18	1.29	1.42	1.56	1.74	1.96	2.24	2.61
60	1.00	1.10	1.20	1.32	1.46	1.63	1.83	2.08	2.41	2.85
65	1.00	1.10	1.22	1.36	1.51	1.70	1.92	2.21	2.59	3.11
70	1.00	1.11	1.24	1.39	1.56	1.77	2.02	2.35	2.79	3.39
75	1.00	1.12	1.26	1.42	1.61	1.84	2.13	2.50	3.00	3.70
80	1.00	1.13	1.28	1.45	1.66	1.92	2.24	2.66	3.23	4.04
85	1.00	1.14	1.30	1.49	1.72	2.00	2.35	2.83	3.47	4.40
90	1.00	1.15	1.32	1.52	1.77	2.08	2.48	3.00	3.74	4.81
95	1.00	1.16	1.34	1.56	1.83	2.17	2.60	3.19	4.02	5.24
100	1.00	1.17	1.36	1.60	1.89	2.26	2.74	3.39	4.32	5.72
105	1.00	1.17	1.38	1.63	1.95	2.35	2.88	3.61	4.65	6.24
110	1.00	1.18	1.40	1.67	2.01	2.45	3.03	3.83	5.01	6.81
115	1.00	1.19	1.42	1.71	2.08	2.55	3.19	4.07	5.39	7.43
120	1.00	1.20	1.45	1.75	2.14	2.66	3.35	4.33	5.80	8.11
125	1.00	1.21	1.47	1.79	2.21	2.77	3.52	4.61	6.24	8.35
130	1.00	1.22	1.49	1.84	2.28	2.88	3.70	4.90	6.71	9.66
135	1.00	1.23	1.52	1.88	2.36	3.00	3.90	5.20	7.22	10.54
140	1.00	1.24	1.54	1.92	2.43	3.12	4.10	5.53	7.77	11.50
145	1.00	1.25	1.56	1.97	2.51	3.25	4.31	5.88	8.36	12.54
150	1.00	1.26	1.59	2.02	2.59	3.39	4.53	6.25	9.00	13.69
155	1.00	1.27	1.61	2.06	2.68	3.53	4.77	6.64	9.68	14.93
160	1.00	1.28	1.64	2.11	2.76	3.68	5.01	7.06	10.41	16.29
165	1.00	1.29	1.66	2.16	2.85	3.83	5.27	7.51	11.21	17.78
170	1.00	1.30	1.69	2.21	2.94	3.99	5.54	7.98	12.06	19.40
175	1.00	1.31	1.71	2.27	3.04	4.15	5.83	8.48	12.97	21.16
180	1.00	1.32	1.74	2.32	3.14	4.32	6.13	9.02	13.96	23.10

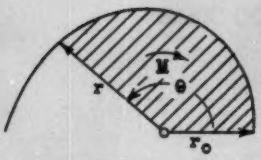
Table B - Area of Sector of Logarithmic Spiral



$$A = \frac{r_o^2}{4 \tan \phi} (e^{2\theta \tan \phi} - 1)$$

θ	Angle of internal friction, φ - degrees									
	0	5	10	15	20	25	30	35	40	45
	Relative area of sector, A/r_o^2									
Degrees										
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	0.04	0.04	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.05
10	0.09	0.09	0.09	0.10	0.10	0.10	0.10	0.10	0.10	0.10
15	0.13	0.13	0.14	0.15	0.15	0.16	0.16	0.16	0.16	0.17
20	0.17	0.17	0.19	0.20	0.20	0.22	0.22	0.23	0.24	0.25
25	0.22	0.22	0.24	0.25	0.26	0.28	0.29	0.30	0.32	0.35
30	0.26	0.27	0.29	0.30	0.32	0.34	0.36	0.38	0.42	0.46
35	0.31	0.32	0.34	0.36	0.38	0.41	0.45	0.48	0.53	0.60
40	0.35	0.37	0.40	0.42	0.45	0.49	0.54	0.59	0.66	0.76
45	0.39	0.42	0.46	0.49	0.53	0.58	0.64	0.72	0.81	0.95
50	0.44	0.47	0.52	0.56	0.61	0.67	0.75	0.85	0.99	1.18
55	0.48	0.52	0.58	0.63	0.69	0.77	0.88	1.01	1.19	1.45
60	0.52	0.57	0.64	0.70	0.78	0.89	1.02	1.19	1.43	1.78
65	0.57	0.63	0.70	0.77	0.88	1.01	1.17	1.39	1.70	2.16
70	0.61	0.68	0.76	0.86	0.98	1.14	1.34	1.62	2.02	2.62
75	0.65	0.74	0.83	0.94	1.09	1.28	1.53	1.87	2.38	3.17
80	0.70	0.79	0.90	1.03	1.21	1.43	1.74	2.16	2.80	3.82
85	0.74	0.85	0.97	1.13	1.33	1.60	1.97	2.49	3.29	4.60
90	0.79	0.90	1.05	1.23	1.46	1.78	2.22	2.86	3.86	5.52
95	0.83	0.96	1.13	1.34	1.61	1.98	2.50	3.28	4.52	6.62
100	0.87	1.02	1.21	1.45	1.76	2.19	2.81	3.75	5.28	7.94
105	0.92	1.08	1.29	1.56	1.92	2.42	3.16	4.28	6.15	9.49
110	0.96	1.14	1.37	1.68	2.09	2.67	3.54	4.89	7.17	11.35
115	1.01	1.20	1.46	1.80	2.27	2.94	3.96	5.57	8.35	13.56
120	1.05	1.26	1.55	1.93	2.46	3.24	4.43	6.33	9.71	16.19
125	1.09	1.32	1.64	2.06	2.67	3.56	4.94	7.21	11.29	19.32
130	1.13	1.39	1.74	2.20	2.89	3.91	5.51	8.20	13.12	23.06
135	1.18	1.45	1.84	2.35	3.12	4.28	6.14	9.30	15.24	27.50
140	1.22	1.52	1.94	2.51	3.37	4.69	6.83	10.56	17.69	32.79
145	1.26	1.59	2.04	2.68	3.64	5.13	7.60	11.98	20.52	39.08
150	1.31	1.66	2.15	2.86	3.92	5.62	8.45	13.59	23.81	46.58
155	1.35	1.73	2.26	3.04	4.22	6.14	9.40	15.41	27.61	55.50
160	1.40	1.80	2.38	3.23	4.54	6.70	10.45	17.45	32.01	66.13
165	1.44	1.87	2.50	3.43	4.89	7.32	11.60	19.75	37.10	78.76
170	1.48	1.94	2.62	3.64	5.25	7.97	12.88	22.40	43.00	93.83
175	1.53	2.02	2.74	3.86	5.64	8.69	14.28	25.30	49.83	111.8
180	1.57	2.09	2.87	4.09	6.06	9.49	15.84	28.65	57.74	133.1

Table C - Moment of Sector of Logarithmic Spiral



$$M = \frac{r_o^3}{3+27\tan^2\phi}\left[e^{3\theta\tan\phi}(\sin\theta + 3\cos\theta\tan\phi) - 3\tan\phi\right]$$

θ	Angle of internal friction, ϕ - degrees									
	0	5	10	15	20	25	30	35	40	45
Degrees	Relative moment of sector, M/r_o^3									
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03
10	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.08
15	0.09	0.09	0.09	0.10	0.10	0.10	0.11	0.11	0.11	0.13
20	0.11	0.12	0.13	0.13	0.14	0.15	0.16	0.17	0.18	0.20
25	0.14	0.15	0.16	0.17	0.18	0.19	0.21	0.23	0.25	0.29
30	0.17	0.18	0.19	0.20	0.22	0.24	0.27	0.30	0.34	0.40
35	0.19	0.21	0.22	0.24	0.27	0.30	0.33	0.38	0.44	0.53
40	0.21	0.23	0.26	0.28	0.31	0.35	0.41	0.47	0.56	0.70
45	0.24	0.26	0.29	0.32	0.36	0.41	0.48	0.58	0.70	0.89
50	0.26	0.28	0.32	0.36	0.41	0.48	0.57	0.69	0.86	1.12
55	0.27	0.31	0.35	0.40	0.46	0.54	0.63	0.81	1.04	1.41
60	0.29	0.33	0.37	0.43	0.51	0.61	0.74	0.94	1.23	1.72
65	0.30	0.35	0.40	0.46	0.54	0.67	0.83	1.07	1.44	2.07
70	0.31	0.36	0.42	0.49	0.59	0.73	0.92	1.20	1.66	2.45
75	0.32	0.37	0.44	0.52	0.62	0.78	0.99	1.33	1.87	2.84
80	0.33	0.38	0.45	0.54	0.65	0.82	1.06	1.43	2.06	3.20
85	0.33	0.39	0.46	0.55	0.67	0.85	1.10	1.51	2.20	3.48
90	0.33	0.39	0.46	0.55	0.68	0.86	1.12	1.54	2.26	3.60
95	0.33	0.39	0.46	0.55	0.67	0.85	1.10	1.50	2.15	3.43
100	0.33	0.38	0.45	0.53	0.65	0.80	1.03	1.29	1.91	2.80
105	0.32	0.37	0.43	0.51	0.60	0.72	0.89	1.09	1.33	1.44
110	0.31	0.36	0.41	0.47	0.53	0.60	0.66	0.64	0.34	-1.01
115	0.30	0.34	0.38	0.41	0.44	0.43	0.32	-0.05	-1.24	-5.04
120	0.29	0.32	0.34	0.34	0.31	0.19	-0.14	-1.05	-3.58	-11.38
125	0.27	0.29	0.29	0.27	0.15	-0.12	-0.78	-2.45	-6.99	-20.93
130	0.26	0.26	0.23	0.15	-0.05	-0.51	-1.61	-4.35	-11.81	-34.97
135	0.24	0.22	0.17	0.03	-0.29	-1.01	-2.69	-6.88	-18.46	-55.3
140	0.21	0.18	0.09	-0.12	-0.59	-1.63	-4.07	-10.21	-27.51	-83.9
145	0.19	0.14	0.00	-0.29	-0.93	-2.38	-5.77	-14.49	-39.58	-124.0
150	0.17	0.09	-0.09	-0.49	-1.34	-3.27	-7.90	-19.96	-55.7	-179.3
155	0.14	0.04	-0.20	-0.71	-1.81	-4.34	-10.48	-26.89	-76.6	-254.9
160	0.11	-0.02	-0.31	-0.95	-2.35	-5.49	-13.63	-35.57	-103.9	-349.4
165	0.09	-0.08	-0.44	-1.22	-2.97	-7.06	-17.39	-46.33	-139.9	-494.
170	0.06	-0.14	-0.57	-1.52	-3.66	-8.74	-21.85	-59.48	-183.8	-677.
175	0.03	-0.20	-0.71	-1.85	-4.43	-10.69	-27.19	-75.63	-240.1	-917.
180	0.00	-0.27	-0.86	-2.20	-5.29	-12.91	-33.39	-95.13	-311.1	-1232.

Discussion of
"BASIC CONCEPTS ON THE COMPACTION OF SOILS"

by C. Y. Li
(Proc. Paper 862)

E. J. ZEGARRA,[1] A.M. ASCE.—A commendable effort has been made by Mr. Li to apply a simplified analysis based on the principle of conservation of energy to the compaction of soil masses as used in construction of earthworks. It is possible that such approach may eventually lead to a rational and unique solution of this confused subject. In spite of the shortcomings and perplexities inherent in the use of empiricism to daily and immediate decisions the field man has no other recourse. And he has no other recourse because the office engineer and the laboratory technician have been unable to duplicate a method of compaction that would in all respects approach field conditions. At this time the laboratory does not have a prototype test for field compaction by modern equipment. For practical purposes, field compaction is a process of reducing the volume of a mass of loose soil with infinite boundaries to obtain strength and imperviousness. Such boundaries cannot be duplicated in the laboratory where compaction is achieved by impact blows of a hammer delivered to a small mass of soil contained within rigid walls of a mold.

The action of a foot in a roller or of a heavily loaded wheel is of relative short duration but it is not an impact. In this respect it resembles the action of driving a sampling tube into the ground by static pressure as differing from that of driving the same sampler by blows of a hammer. The resistance of the soil to these actions may be appreciated by examination of Fig. 1A. It is seen in such graph that only when the velocity has reached about 90 percent of the maximum, as at impact, that the resistance approaches a peak. Anything short of impact meets with static resistance. At the end of impact roughly 40 percent of the settlement has taken place. Thus a static load would need to be of very large magnitude to equal the results achieved by impact. Here lies the difference between laboratory and field compactive efforts and also the difference between hammer compaction and static compaction as exemplified by the Proctor and Harvard miniature laboratory methods.

A good deal of work has been done towards correlation of the two compactive efforts with a view towards specifying the number of passes, type and weight of roller that would reproduce a density and strength equal to that obtained in the laboratory. For several reasons the correlation has not been effective. At one time the Standard compaction method was generally used to define the optimum moisture content and maximum density to be obtained in the field. Industry, always with an eye towards economy in contracts, soon devised machines that would compact soils more efficiently and produced masses of higher density than those specified. As a result of these performances it was usual to specify densities from 105 to 110 percent of those

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obtained in the laboratory. A modification of the Standard compaction method became necessary and was put into effect by the AASHO and other agencies. Modification consisted of changes in the dimensions of some of the factors, e.g. weight of hammer, height of drop, thickness of layer. Other tests based on compaction by static energy were also devised (Wilson, Osterberg, Porter) which resulted in better correlation with field densities. These tests, although more realistic, have not been universally adopted. There is yet much research to be done before specifications strictly based on laboratory tests can be adhered to in the field. At this time there are indications that the field compaction effected by modern machinery is capable of producing higher densities than those obtained by common laboratory tests. Plainly, the root of the discrepancy has not been exposed and it seems unlikely that a theoretical simile would yield the answer.

Another factor which deserves a modified outlook is that of molding water content. An investigation to determine the optimum area of a foot in a roller of given weight was made by the Waterways Experiment Station and reported by S. J. Johnson and W. G. Shockley, members ASCE,[1] to take into account the heavier pressures exerted by new sheepsfoot rollers employed during the last 15 years. One of the conclusions reached in that research was that a foot size could be selected that would allow the roller to "walk out" of the soil when the water content was above optimum. It can be argued conversely, that a water content could be selected that would allow any practical size of foot to "walk out." This points to the shear strength which permits the soil to fail under a given pressure if the water content is higher than the optimum for a given compactive effort. Soils are intricate complexes of physical and mineralogical properties occurring in nature in myriad variations. The water absorbing characteristics of fine grained soils affect their strength. It would be odious therefore to restrict the compactive effort because of a limiting water content. If that were the criterion there would be limitations to the strength of the soil with a consequent increase in the volume required. A laboratory test performed under given conditions should serve as a measure for the results to be obtained in the field.

Stage compaction, as advocated by Mr. Li, would tend towards the relaxation of field control of moisture. If a soil, as pointed out in the paper, had a moisture content higher than optimum and therefore a lower potential shear strength, a lighter roller would be employed. This would be the obvious means of compacting such soil and would, if used long enough, decrease the moisture content by aeration, then compaction would be shifted to a heavier roller and so on. No laboratory test would ever, under such conditions be of any value.

The need to take account of the results of field compaction into laboratory tests is urgent if economies of design are to be achieved. Field forces are notoriously independent and either reluctant or neglectful in reporting compaction results in detail and frequency. As a consequence of these conditions the laboratory continues to use a method that may either be too lenient or too strict. Of course a minimum specification would always be required and, for the time being, the modified AASHO compaction method appears to be only a little behind modern machinery and field methods of construction. As data

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1. Field Penetration Tests for Selection of Sheepsfoot Rollers S. J. Johnson, M. ASCE, W. G. Shockley, A.M., ASCE, Separate 363, Vol 79, ASCE.

from the field are collected and a variety of soils is reported, the laboratory methods would be revised. This process requires the cooperation and understanding of the field man, the office engineer and the policy maker all of whom are concerned with the construction of earthworks.

WALTER A. BROWN,[1] A.M. ASCE.—The paper by Mr. Li is a very thoughtful and lucid development of basic concepts of the soil compaction process and should do much to clarify the thinking of engineers responsible for specifying compaction methods.

The concept of "stage compaction" is of particular interest and seems worthy of general adoption on larger work. It is not uncommon, where heavy pneumatic rollers are being employed on cohesionless sandy soils, to observe that the heavy rollers are unable to travel at all until some first stage compaction is achieved by walking a tractor over the material or applying water. This, of course, is an extreme case but is illustrative of the wasted energy without precompaction. It is a commonly accepted idea that a sheepsfoot roller is not fully effective unless it "walks out." In such instances, instead of simply resorting to a roller having a smaller pressure intensity, the use of stage compaction may permit the use of the heavier roller in the final stage of compaction with resultant better densities.

The reminder that "optimum moisture content" of a soil type is that for one compactive effort only, is timely. Particularly on smaller earth dam construction where heavy compaction equipment is not available, it has been found that densities corresponding to compactive efforts much larger than that used in the standard AASHO laboratory test are seldom obtained in practice. An optimum moisture obtained in the laboratory from the modified AASHO test, having nearly five times the compactive effort of the standard AASHO test, is obviously far from the correct optimum moisture for field compaction in these cases. For this reason it is suggested that an arbitrary laboratory compaction standard as near as possible to the results found obtainable in the field be used. The control requirement should then be a per cent compaction somewhere near or above 100% of this arbitrary laboratory standard which need not conform in compactive effort to any of the various published standards. For more important work with conventional heavy rollers, it is believed that a standard using a compactive effort of about 20,000 foot pounds per cubic foot is generally consistent with the attainable field compaction.

The influence of gravel content on compaction is encountered very frequently now that zoned dams employing semi-pervious or pervious shells are becoming commonplace. The problem of a satisfactory method of density control is a difficult one when the quantity of plus No. 4 sieve sizes exceeds about 20%. When the quantity of plus No. 4 is less than this amount, the standard density test has been found generally satisfactory using the rock correction as set forth in the author's equation (7). Beyond this point, however, no well-known standardized method has been developed except the "relative density" control used by the U. S. Bureau of Reclamation for free-draining materials.

If low permeability is not a requirement, it is thought that the presence of gravel in a soil compensates to a large degree in strength for the lower density and hence less strength achieved in the minus No. 4 fraction of the

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material. In many cases it is necessary in the interest of economy to use dirty gravels or a soil-rock mixture in the outer zones of a dam instead of grizzlying or otherwise processing the material. The "relative density" method of control of such materials is difficult to apply. First the material may not be sufficiently free draining to obtain a laboratory "maximum" density by vibration and flooding. Second, even if the foregoing is possible, the method is laborious and time consuming in that a "maximum" and "minimum" laboratory density must be obtained for each field test.

In instances of this kind, a method of control is suggested which has been used with reasonable success. This method is to develop in the laboratory a curve similar to the curve labeled "Actual Total Density" on the author's Figure 6. The curve is developed by compacting, at optimum moisture of the fines and at a compactive effort comparable to attainable field compaction as described above, samples of the material using various percentages of plus No. 4 material. A large compaction cylinder is required, preferably one cubic foot in size, with a correspondingly large compaction hammer. Once having obtained this basic laboratory curve for the material, field densities can be checked against it after having determined the dry density of the field sample and the per cent of plus No. 4 material in the sample. One or two such basic curves are usually sufficient if the material comes from a single source of borrow. As described above, control may be exercised by requiring a percentage compaction at or near 100% of the densities shown by the laboratory curve.

It is thought that Mr. Li's paper provides a guide to further experimentation and rational development of compaction methods.

Discussion of
"PENETRATION TESTS AND BEARING CAPACITY OF COHESIONLESS SOILS"

by G. G. Meyerhof
(Proc. Paper 866)

NAI-CHEN YANG,[1] A.M. ASCE.—The scope of this discussion is confined to the bearing capacity of piles and piled foundation. As mentioned in the paper, both the point resistance and skin friction of a pile varied from the resistance of a penetrometer. According to Seed and Reese and the writer,(1,2) the fluctuation of pile resistance is primarily due to the post-driving stress adjustment in the soils encountered by a pile. The physical properties of the soils as well as the time interval of stress adjustment respond the degree of fluctuation. As the resistance of a penetrometer resembles more or less the resistance of a pile at driving, it is doubtful that the average value of some observations, as expressed by the Equations (14a) and (15), are applicable for the piles in a subsoil varied from gravelly sand to sandy silt. Moreover, according to Fig. 4, the widely scattered plots do not suggest any fast relationship between skin friction and average static cone resistance. The application of Eq. (16b) is, therefore, very arbitrary and may mislead the compilation as shown in Table 3.

Insofar as the piled foundation is concerned, an actual job case is worth noting. The foundation under discussion is located in Brooklyn, N. Y., where shore deposits, predominately of fine sand, are encountered to a great depth. The loose density of the sand deposit called for the use of friction piles, similar to the Franki type, spaced at about 3 to 3-1/2 pile diameters. Standard penetration tests were carried out both before and after the pile driving (see Fig. 5). The increasing resistance of standard penetration by compaction of piles have an effect to offset the practical value of the Equations (20b) and (23), in which no clarification is made on the N-value, either before or after the pile driving. However, it seems that the Equation (25) may give a fair approximation by using the N-value before pile driving. It is the writer's opinion that a different philosophy should be approached toward the bearing capacity of a single pile or a piled foundation.

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2. Yang, Nai-Chen, (1956) Redriving Characteristics of Piles, Proceedings, ASCE (Submitted for publication).

1. Foundation Engr., Madigan-Hyland, New York, N. Y.

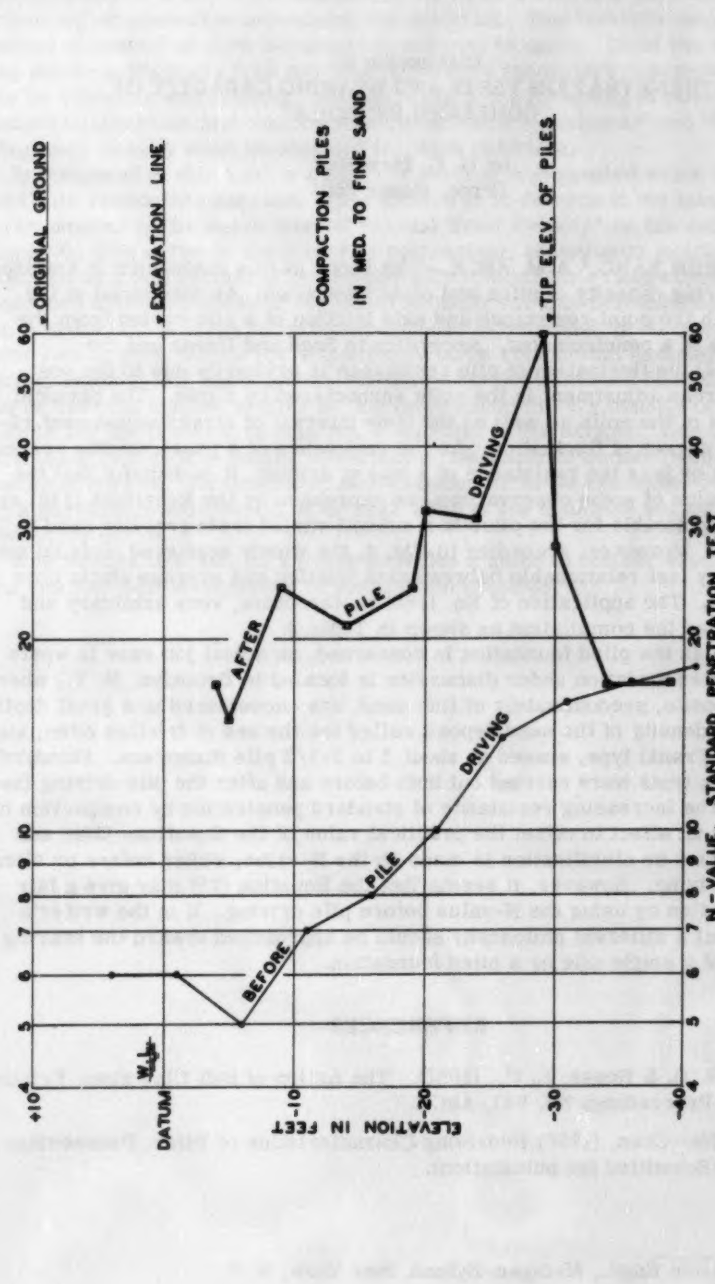

Fig. 5 Change of N-Values due to Pile Driving

SUMNER G. HYLAND,[1] J.M. ASCE.—The author is to be congratulated for his excellent paper. The writer, however, would like to discuss a procedure dismissed rather briefly by the author, namely, undisturbed sampling in cohesionless soils.

The procedure of taking undisturbed samples below the water table in cohesionless soils has been most often dismissed as difficult, if not impossible. The undisturbed sampling, however, may be carried out at relatively little expense provided the procedure of "mud" drilling is used. The use of "mud" drilling in taking these samples is discussed in detail in the Corps of Engineers Waterways Experiment Station Publication titled "Undisturbed Sand Sampling Below the Water Table." The procedure briefly consists in using a heavy slurry of "drilling mud" to restrain the soils of the hole in lieu of casing. The soil and water forces on the walls of the hole are counter-balanced, or nearly so, by the weight of the slurry to keep the walls of the hole from squeezing in. This makes the common phenomenon of sand flowing into a cased hole because of artesian conditions impossible. Either piston samples or standard penetration samples may be taken of the soil below the water table in its natural state. The piston sample once taken may be used for relative density determinations.

An undisturbed or standard penetration split-spoon sample taken in a hole drilled with "mud" instead of casing eliminates two basic faults with the usual method of driving casing and cleaning it out with clear water. One of these faults is that the driving of the casing tends to densify a loose cohesionless soil. This produces a high standard penetration resistance and a higher natural density from an undisturbed sample.

Another is that artesian water conditions in a cased borehole drilled with clear water may also produce a quick condition in the bottom of the hole which produces the phenomenon known as "running sand." This produces lower natural densities from an undisturbed sample and lower standard penetration resistance.

The writer has noted several instances in which a comparison between the standard penetration resistance and the relative density of the cohesionless soil when sampled with a piston sampler have been at wide variance; in particular, a case in which the penetration resistance was one blow per foot, yet the relative density of the undisturbed soil was of the order of 90-95%. An occurrence such as this, while not the rule, may happen. Many factors other than relative density may affect penetration resistance, such as size and shape of particle, amount of clay and length of drill stem between sampler and drive hammer.

In summary, the writer believes that there are many cases in which undisturbed sand sampling can be carried out at little more than the cost of ordinary undisturbed sampling. He also believes that the standard penetration resistance would be far more valuable in cohesionless soils below the water table if they were drilled with mud and not with clear water and casing.

L. J. MURDOCK.[2]—The writer's organization has found the standard penetration test more convenient to perform than the static cone penetration test, since the former can be rapidly and conveniently carried out in an exploratory borehole, whereas the latter requires cumbersome apparatus held down

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2. Mgr., Central Lab., Geo. Wimpey & Co., Ltd., Southall, Middlesex, England.

by kentledge or earth anchors. Also the presence of layers of gravel or cemented sands can cause refusal of further penetration of the static cone.

This firm has only made a few comparisons between standard penetration and cone resistance in cohesionless soils. On two sites in alluvial deposits in Iraq, the average of eight comparisons of standard penetration to cone resistance gave

$$q_c = 5.9\ N$$

where q_c = static cone resistance (tons/sq.ft)

at N = standard penetration resistance (blows/ft. penetration)

A difficulty in using the results of any form of penetration test for determining the skin friction and end resistance on driven piles in cohesionless soils, is the effect of the pile driving on the density of the soil. Driving piles in large groups tends to compact such soils, thus increasing the driving resistance and, hence, the carrying capacity of individual piles.

It would be useful to know whether the author's method takes this effect into consideration; or is the allowable bearing pressure on a group of piles governed solely by considerations of consolidation settlement of the soil strata below pile point level which are presumably unaffected by the pile driving?

W. J. TURNBULL,[1] M. ASCE, and R. I. KAUFMAN,[2] J.M. ASCE.—The correlation between estimated bearing capacity of shallow and pile foundations with standard penetration tests is of considerable interest. The writers have been associated with several projects where the standard (dynamic) penetration test in which a 2-in. OD by 1-3/8-in. ID split spoon sampler driven under an energy of 350 ft-lb has been used. This penetration test was used in the field investigation of the sites for the Morganza Floodway control structure, Morganza, La., and the low sill control structure at Old River, La. Both structures were to be founded on displacement piles, and pile loading tests were performed to determine the required size and length of piles. The writers' discussion is related to comparisons between the observed pile capacities at these two structures and those estimated from the author's equations using the standard dynamic penetration test.

The relationships for estimating bearing capacity from penetration resistances are based on the data given in Table 1 of the paper, which assumes a relationship between the angle of internal friction of a cohesionless soil and the standard penetration resistance. The validity of this relationship depends on numerous factors, including the inertia of the drill rods, and effective overburden pressure at the depth at which the penetration test is performed. The overburden pressure was found to have a large influence on penetration resistance at the site of the proposed low sill structure.

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1. Chief, Soils Div., Waterways Experiment Station, Corps of Engrs., U. S. Dept. of the Army, Vicksburg, Miss.
2. Chief, Design and Analytical Section, Embankment and Foundation Branch, Soils Div., Waterways Experiment Station, Corps of Engrs., U. S. Dept. of the Army, Vicksburg, Miss.

During the site investigation at the low sill structure, a standard split spoon boring was made at the original ground surface. Soil conditions at the site are illustrated in figure A. As the structure is to be founded in an excavation about 50 ft. deep, pile loading tests were conducted in a test excavation of this depth, to eliminate the effect of the overburden above the base of the structure on the pile capacities. After the excavation was to grade, a split spoon boring was made in the bottom of the excavation to determine the effect of the removal of 50 ft. of overburden on the split spoon driving resistance. A comparison between the split spoon driving resistance for both borings is shown in Figure A and indicates that the removal of 50 ft. or overburden reduced the split spoon driving resistance by one-half in the silty soils between the bottom of the excavation (el 0) and el -50. A decrease in the driving resistances in the deep sands below el -50 also was noted. Unfortunately, in the boring made from the ground surface, the exact resistance in the sand was not determined whenever it exceeded 100 blows per ft., so an exact comparison cannot be made for the sand stratum.

The split spoon driving resistance for the upper portion of the sand in which the test piles were founded (el -48 to -81) was about 75 to 100 blows per ft. (equivalent resistance of about 45 to 57 blows per ft., as defined by the author). According to Table 1, the estimated angle of internal friction would be at least 45 degrees for either the standard or equivalent penetration resistance. However, laboratory triaxial shear tests on undisturbed samples of the sand below el -50 indicated an angle of internal friction of about 33 to 36 degrees.

Soil conditions at the Morganza Floodway control structure, in order of depth, consisted of about 80 ft. of plastic clay underlain by a very thick stratum of clean sand. The standard split spoon driving resistance in the upper portion of the sand stratum (in which the piles were founded) was about 35 to 70 blows per ft. and averaged about 55 blows per ft., corresponding to an average equivalent resistance of about 35 blows. According to Table 1, the estimated angle of internal friction of the sand would be about 45 degrees based on the standard resistance, or about 40 to 45 degrees based on the equivalent resistance. Analyses of pile loading test data indicated the angle of internal friction to be about 30 to 35 degrees.

In view of the above differences between the angle of internal friction of sand as estimated from Table 1 and those determined either by laboratory tests or pile loading tests, it appears that the relationships given in the author's Table 1 should be used with care, especially since theoretical bearing capacity factors increase rapidly for values of ϕ in excess of about 30 degrees.

Comparisons between the observed ultimate bearing capacity of test piles at the Morganza Floodway control structure and the low sill control structure for Old River, and those estimated from the author's equation (18-b), have been made by the writers. Although the sand strata at both sites were composed of relatively clean fine to medium sands, the equivalent penetration resistances normally assigned to silty or very fine sands were used instead of the standard penetration resistances, as the former gave results that agreed slightly better with the observed data. At the Morganza Floodway control structure,[3] the ultimate bearing capacity in sand was determined by testing

3. Mansur, C. I. and Focht, J. A., "Pile Loading Tests, Morganza Floodway Control Structure." Proceedings, ASCE, Separate No. 324, November 1953.

seven pairs of piles to failure. One pile of each pair was driven into sand and the other was founded in clay just above the sand so that it would act as a skin friction pile. The ultimate bearing capacity for the portion of the pile driven into sand was taken as the difference in failure load of the two piles comprising the pair, and is given in table A. Also given in this table are the estimated ultimate bearing capacities as obtained from the author's equation (18-b). The observed average capacity of the piles was about 65 per cent of that estimated by this equation, with considerable variation occurring in individual values.

At the low sill structure, where the pile loading tests were conducted in an excavation, the piles were equipped with strain rods at 5- to 15-ft. intervals along the length of the pile, so that the distribution of skin friction along the pile and the point resistance could be determined. The observed average values of unit skin friction for the portion of the pile embedded in sand and unit tip capacity are given in table B. Also given in this table are the estimated unit tip bearing capacity and unit skin friction as determined from the author's equation (14-b) and (16-b), respectively. The average of the unit skin friction for the piles tested was only about one-half of that estimated by means of the author's formula (16-b); and the actual unit tip capacity was only about 24 per cent of that estimated from the author's equation (14-b). Also of interest is the fact that the 14-in. H-beam acted as a 14-in.-square displacement pile, in so far as unit point bearing capacity was concerned, because the unit point bearing capacity was at least equal to that observed for the displacement type test piles.

The results of the above-mentioned pile loading tests indicate that the bearing capacity of a pile estimated from driving resistance of a split spoon sampler using the author's equations was considerably greater than the observed bearing capacity. Had the bearing capacities of the piles been estimated using the actual standard penetration resistances instead of the lower equivalent resistances, the estimated capacities would have been even less conservative than those shown in tables A and B. Therefore it is believed that bearing capacities estimated by means of the author's equations should be used with care when designing pile foundations. The writers consider it preferable to perform laboratory tests on undisturbed samples of sand to determine the angle of interval friction and then use theoretical formulas for estimation of pile capacity. This procedure was followed for the purpose of determining a preliminary pile design for the low sill structure. The bearing capacity of the sand was computed from Terzaghi's[4] formulas, which admittedly may not be exactly applicable to piles but which, nevertheless, gave estimated values which checked closely with the observed capacities of the test piles. A summary of the computed and observed bearing capacities obtained from pile loading tests at the low sill structure is given in table C.

Again, the writers wish to emphasize that the relationships given in Table 1 depend on length of drill rods, overburden pressure, elevation of ground-water table, stress history of soil, and other factors which are difficult to evaluate, and that the indicated relationship between the standard penetration resistance and the angle of internal friction of the soil and corresponding bearing capacity should be used with care when designing pile foundations.

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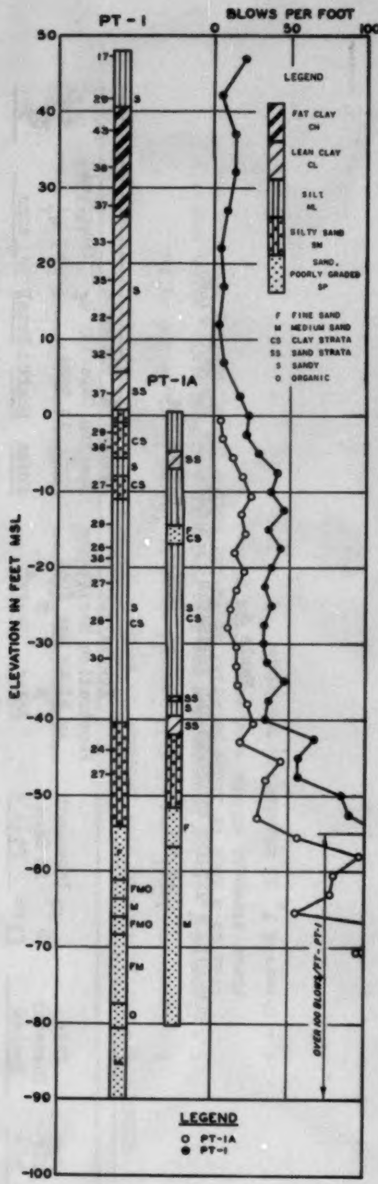
4. Terzaghi, Karl, Theoretical Soil Mechanics. John Wiley & Sons, New York, N. Y., 1934.



NOTE: DRIVING RESISTANCES (BLOWS PER FOOT) DETERMINED WITH A STANDARD SPLIT SPOON SAMPLER (1-3/8-IN. ID, 2-IN. OD) AND A 140-LB HAMMER DROPPED 30 INCHES.

BORING ADVANCED WITH FISHTAIL USING DRILLING MUD.

FIGURES TO LEFT OF BORINGS ARE NATURAL WATER CONTENTS IN PER CENT DRY WEIGHT.

BORING PT-1 MADE FROM GROUND SURFACE IN AUGUST 1954. BORING PT-1A MADE FROM BOTTOM OF EXCAVATION (ELEV 0) IN FEBRUARY 1955.

BORING CLASSIFIED IN ACCORDANCE WITH THE UNIFIED SOIL CLASSIFICATION SYSTEM USED BY THE CORPS OF ENGINEERS, U.S. ARMY.

FIG. A. SPLIT SPOON BORING DATA, LOW SILL STRUCTURE, OLD RIVER, LA.

Table A

COMPARISON BETWEEN OBSERVED AND ESTIMATED PILE CAPACITY FOR TEST PILES AT THE MORGANZA FLOODWAY CONTROL STRUCTURE, LA.

Pile No.	Pile Diameter inches	D ft	Imbedment in Sand ft	Equivalent Penetration Resistance in blows per foot		Estimated Ultimate Bearing Capacity Q_f in tons			Observed Q'_f tons	Ratio $\frac{Q'_f}{Q_f}$
				$\bar{N}$ Point	$\bar{N}$ Shaft[a]	Point	Shaft	Total		
C-1-b	24	80	9.0	32.5	20	408	23	431	213	0.49
C-2-b	8 (tip)	90	7.6	28.5	32.5	40	10	50	85	1.70
C-3-b	12	85	1.9	45	40	142	5	147	88	0.60
C-4-b	18	84	3.5	42.5	27.5	301	9	310	163	0.53
C-5-b	24	89	5.7	25	37.5	314	27	341	168	0.49
C-6-b	30	84	6.1	42.5	30	835	29	864	286	0.33
C-7-b	22 (square)	95	12.8	25	36	336	58	394	152	0.39
									Average	0.65

[a] For portion of pile imbedded in sand.

Table B

COMPARISON BETWEEN OBSERVED AND ESTIMATED UNIT POINT RESISTANCE AND UNIT SKIN FRICTION FOR TEST PILES AT THE PROPOSED LOW SILL STRUCTURE, OLD RIVER, LA.

Pile No.	Pile Diameter inches	D ft	Equivalent Penetration Resistance in blows per foot: N Point	$\bar{N}$ Shaft[a]	q_p in tons/sq ft: 4N	Observed	Ratio: Observed to Estimated q_p	f_s in tons/sq ft: $\bar{N}/50$	Observed	Ratio: Observed to Estimated f_s
1	14 H-beam	81	57.5	47.5	230	68[b]	0.30	0.95	0.31	0.33
2	21	65	45	37.5	180	31	0.17	0.75	0.42	0.56
3	14 H-beam	71	57.5	45	230	37[b]	0.16	0.90	0.10[c]	--
4	17	66	45	37.5	180	48	0.27	0.75	1.84[d]	--
6	19	65	45	40	180	50	0.28	0.80	0.43	0.54
						Average	0.24		Average	0.48

NOTES: a - For portion of pile imbedded in sand.

b - Cross-sectional area of pile assumed equal to 14- by 14-in. = 196 sq in.

c - Observed f_s is low because pile was equipped with a bottom plate, which probably created a zone of loose material above the bottom plate, thereby reducing the shear strength of the sand around the pile.

d - Observed f_s is considered too high.

Table C

RELATIONSHIP BETWEEN OBSERVED AND COMPUTED BEARING CAPACITY OF PILES FOUNDED IN SAND, LOW SILL STRUCTURE, OLD RIVER, LA.

Test Pile No.	Ultimate Bearing Capacity* in tons: Q'_f Observed	Q_f ** Computed	Ratio: Q'_f/Q_f
1	185	210	0.88
2	168	175	0.96
3	136	155	0.88[a]
4	200	130	1.54[b]
6	136	150	0.91
		Average	0.92

NOTES:

* For portion of pile imbedded in sand.

** Computed Q_f based on: $K = 1.0$, $\phi = 33°$, and Terzaghi's bearing capacity factors.

a - Results not included in average, as skin friction was considered too low; see table B.

b - Results not included in average, as skin friction was considered too high; see table B.

JOHN A. FOCHT, JR.,[1] A.M. ASCE.—Prof. Meyerhof has presented procedures by which the bearing capacity of spread footings and piles may be estimated from the results of standard 2-inch O.D. split-spoon penetration tests. The engineer who considers using these suggested procedures might well note the number of times the author used the word "approximate," "estimate" and their derivatives—13 for "approximate" and 18 for "estimate." The liberal usage of these descriptive words offers significant warning that the proposed relationships are not hard and fast rules. The author is apparently well aware of the limitations of the procedures but others might not be as well informed.

A danger exists in the introduction of very approximate procedures for estimating bearing capacity, particularly those based on the dynamic resistance of some instrument to penetration. Take for example the well-known Engineering News pile driving formula. Although the literature is full of reports establishing beyond all doubt that the validity of the formula is very poor, it still enjoys a widespread use and is sometimes accepted as very precise. A simple expression which can easily be used, even though originally published with statements pointing out its many limitations, may soon become the standard to be applied to all situations without regard to its applicability. Therefore, caution is warranted in promulgating or in using equations which apply results of dynamic penetration tests to foundation design.

The difficulty of securing a "close correlation between relative density and penetration resistance (of a standard 2-inch spoon)" is mentioned. But the wide range of relative densities that may correspond to a single value of N was not stated in quantitative terms. On table 1 (writer) are listed the

TABLE 1

Overburden Pressure psi	Relative Density for N = 10*			
	Fine Sand		Coarse Sand	
	Dry	Saturated	Dry	Saturated
0	75	-	73	82
20	47	78	42	68
40	40	72	31	47

*Taken from curves in Reference 1

relative densities corresponding generally to a resistance of 10 blows per foot as shown by tests by the U. S. Bureau of Reclamation.(1) A value of N = 10 can be developed in a dry coarse sand with a relative density of only 30% if the overburden pressure is 40 psi or in a saturated fine sand with no overburden pressure with a relative density approaching 100%. This is a very wide range. These tests did show however that the approximate relationship(2) between the standard penetration resistance and the relative

1. Chf. Design Engr., McClelland Engrs., Inc., Houston, Tex.

density of sands is generally conservative except for high overburden pressures and high values of N. At low overburden pressures the suggested relationship[2] was found to be quite conservative.

The U.S.B.R. tests performed under laboratory controlled conditions eliminated the personal variation so dominant in the standard penetration test. The soil type, length and type of drill rods, and ground water conditions were mentioned by Prof. Meyerhof as variables affecting the driving resistance. Not mentioned, however, are the large group of factors restricting the "free" fall of the hammer which include the drilling equipment, driller, weather, etc. The multiple and variable combination of all these factors can, and probably oftentimes do, result in variations in N greater than those which would have been observed due to only changes in relative density.

The preceding comments were presented not as specific criticism of the information developed in the paper. They were intended to emphasize that N is not a factor which is a distinct soil property but is a function of several soil properties and other physical conditions. While the equations presented based on N can serve as guides to the design of foundations on cohesionless soils, it should be noted that, although in some instances the equations will produce very conservative estimates, in other instances the estimates obtained are not conservative.

Table 3 (author) purports to show the general validity and conservatism of Eqs. 14b and 16b for the bearing capacity of piles. However, not all of the pile data from one of the instances[3] cited in this table were presented. These omitted data given in Table 2 (writer) show that the pile capacity estimated by Eqs. 14b, 16b and 18b are very unconservative, overestimating the capacity of the piles in sand by as much as 5 times. These piles were driven through about 80 feet of soft clay into dense fine sand. Therefore, in using

TABLE 2

Pile No.	Pile Size B feet	Pile Embedded Length D feet	Average Pentr. Resistance: Point N blows/ft	Average Pentr. Resistance: Shaft $\bar{N}$ blows/ft	Ultimate Bearing Capacity in Sand: Observed Q'_f tons	Estimated: Point tons	Estimated: Shaft tons	Estimated: Total Q_f tons	Q'_f/Q_f
C1b	2.0 dia.	9.0	55	39	176	691	44	735	0.24
C2b	0.8 dia.	7.6	48	51	74	68	17	85	0.87
C3b	1.0 dia.	1.9	70	62	85	220	6	226	0.39
C4b	1.5 dia.	3.5	70	62	159	495	16	511	0.32
C5b	2.0 dia.	5.7	50	57	171	880	36	916	0.19
C6b	2.5 dia.	6.1	70	62	276	1373	48	1421	0.20
C7b	1.8 sq.	4.0	50	58	120	672	29	701	0.17

Data taken from Reference 3.

TABLE 3

Pile No.	Size feet	Unit Skin Friction in Silt, f_s Average tons/ft^2	Maximum tons/ft^2	Unit End Bearing in Sand, q_p tons/ft^2
2	1.8	0.65	0.73	31
4	1.4	0.66	1.08	37
5	1.4	0.49	0.65	48
6	1.6	0.76	0.94	50
Average		0.64		42
Estimate by N		0.3 (N = 15)		300 (N = 75)

Results taken from Reference 4.

Eq. 14b and 18b to compute portions of Table 2 (writer), it was assumed that the total penetration was sufficient to produce end bearing corresponding to a penetration ratio of D/B greater than 10, even though the penetration into sand was limited. But even following Eq. 10 to reduce the unit point resistance for the limited penetration into sand, the capacity of some of the piles is still overestimated by 2 times. The use of one value of N to compute the point resistance and another for the shaft resistance would imply that the procedure used in computing Table 2 (writer) is applicable to the condition of piles driven through a considerable thickness of clay into sand.

Recent tests by the Corps of Engineers(4) on other long pipe piles measured the skin friction through considerable silt and in sand, and the end bearing in sand. The vertical movements of a number of points along each pile were carefully observed to provide the necessary data to compute the loads transferred from the pile to the soil by the various strata. The pertinent results of the pile tests are given on Table 3 (writer). The average skin friction in about 50 feet of silt from four pipe piles is 0.64 tons per square foot. The average N in this depth range from standard penetration tests is 15, which by Eq. 16b should indicate a skin friction of 0.3 ton per square foot—a very conservative estimate. The average bearing pressure on the ends of these four piles in sand was 42 tons per square foot. The average N in the sand is in excess of 75; this indicates, by Eq. 14b, a value of q_p greater than 300 tons per square foot which is a gross overestimate. Extending these unit values to estimate pile capacity, the estimated capacities are considerably greater than the test capacities, as much as 3 times greater.

In both of these cases, following the suggested procedures blindly might have produced unsafe pile designs. Although widely accepted, the standard penetration test does not have the precision and reliability attributed to it by many of its users.

REFERENCES

1. "Second Progress Report of Research on the Penetration Resistance Method of Subsurface Exploration" U.S.B.R. Earth Laboratory Report No. EM-356, Oct. 1953.

2. "Soil Mechanics in Engineering Practice" by K. Terzaghi and R. B. Peck, J. Wiley & Sons, New York 1948, p. 294.

3. "Pile Loading Tests, Morganza Floodway Control Structure," C. I. Mansur & J. A. Focht, Jr., Proceedings ASCE, 1953, Vol. 79, No. 324.

4. "Pile Loading Tests, Low-Sill Structure—Old River Control," by C. I. Mansur & R. I. Kaufman, presented to Feb. 1956 ASCE meeting, Dallas, Texas.

Discussion of
"THRUST LOADING ON PILES"

by James F. McNulty
(Proc. Paper 940)

CORRECTIONS.—The following captions should be appended to the illustrations of this paper:

Figure 1. Soil Data, Free-End Concrete Piles

Figure 2. Soil Data and Test Setup, Fixed-End Timber Piles

Figure 3. Load-Settlement Curves, Vertical Load Tests

Figure 4. Load-Deflection Curves, Lateral Load Tests, Free-End Concrete Piles

Figure 5. Lateral Load Test Setup, Free-End Concrete Pile

Figure 6. Lateral Load Test Setup, Fixed-End Timber Piles

Figure 7. Load-Deflection Curves, Lateral Load Test, Fixed-End Timber Piles

Figure 8. Vertical Load Test Setup, Timber Pile

Figure 9. Lateral Deflection of Caps at Conclusion of Lateral Test, Fixed-End Timber Piles

Figure 10. Design Curves for Lateral Loads, Fixed-End Timber Piles, 12-Inch Diameter

Figure 11. Design Curves for Lateral Loads, Fixed-End Concrete Piles, 16-Inch Diameter

Figure 12. Design Curves for Lateral Loads, Free-End Timber Piles, 12-Inch Diameter

Figure 13. Design Curves for Lateral Loads, Free-End Concrete Piles, 16-Inch Diameter

Discussion of
"THRUST LOADING ON PILES"
by [illegible]
(Proc. Paper [illegible])

CORRECTION—The following figures should be appended to the [illegible] of this paper [illegible]:

Figure 1. Soil Data, Free-End Concrete Piles

Figure 2. Soil Data and Test Setup, Fixed-End Timber Piles

Figure 3. Load-Deflection Curves, Vertical Load Tests

Figure 4. Load-Deflection Curves, Lateral Load Tests, Free-End Concrete Piles

Figure 5. Lateral Load Test Setup, Free-End Concrete Pile

Figure 6. Lateral Load Test Setup, Fixed-End Timber Piles

Figure 7. Load-Deflection Curves, Lateral Load Test, Fixed-End Timber Piles

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DIVISION ACTIVITIES
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

NEWSLETTER

July, 1956

DIVISION EXECUTIVE COMMITTEE
OCTOBER, 1955 - OCTOBER, 1956

		Contact member for Committees on
S. D. Wilson (Chairman)	Shannon & Wilson, 2208 Market St., Seattle 7, Washington.	Earth Dams
W. J. Turnbull	Waterways Exp. Station, Vicksburg, Miss.	Road & Airfield Soil Problems
R. B. Peck	113 Talbot Lab., Univ. of Illinois, Urbana, Illinois	Eng'g. Geology; Technical Sessions
R. E. Fadum	Dept. of Civil Eng'g., North Carolina State Coll., Raleigh, N. C.	Glossary; Grouting
S. J. Johnson	Moran, Proctor, Mueser and Rutledge, 420 Lexington Avenue, New York, N. Y.	Publications
J. O. Osterberg (Secretary)	Dept. of Civil Eng'g., Northwestern University, Evanston, Illinois.	

CURRENT RESEARCH AND DEVELOPMENT

Following is the concluding installment of the material contributed by Mr. R. F. Legget on activities in soil mechanics and related fields on the part of the National Research Council of Canada. The first installment appeared in the April Newsletter.

Associate Committee on Soil and Snow Mechanics, National Research Council, Canada

Soil Mechanics

It has been possible for the Committee to bring all the active workers in this field throughout Canada into touch with one another through the holding of annual Soil Mechanics Conferences. The first meeting of this kind was held in 1947 in Ottawa. A pattern has been established of holding every third conference in Western Canada with the intervening conferences in Ottawa. These meetings provide opportunity for a review of soil mechanics progress during the year and for a full discussion of technical problems which have arisen in Canadian work. The Committee publishes summary records of the meetings.

Arising from the annual conference, local groups that meet monthly have developed in Toronto, Ottawa, Vancouver, and Montreal. The Committee hopes to assist work in the Maritime Provinces and in the West of Canada by arranging for visiting lecturers and special group meetings. A First Maritime Regional Conference was held in Fredericton in the spring of 1954, as a start in the direction of such regional activities.

Canadian Landslides

Closely allied to the Subcommittee on Soil Mechanics, a small subcommittee was established in 1954 to study Canadian landslides, particularly those occurring in clays. This subcommittee will study the distribution of landslides and the written accounts of past landslides. It is hoped that criteria can ultimately be established by which landslips may be forecast in time to take adequate prevention measures.

Muskeg

When the Committee first looked into the problem of muskeg, it encountered great diversity of opinion as to what really constituted this unusual material. After much preliminary study, it was found that the problem could only be solved by a basic approach from the point of view of palaeobotany. The Committee therefore initially subsidized the start of work in this field through the Department of Botany at McMaster University, Hamilton. The Defence Research Board later became associated with the work and provided material assistance for field studies of muskeg in the Churchill district. Study of this organic material is a new phase of scientific inquiry, palaeovegetography. The Committee looks forward to its steady development, now that the work has been so well started, and to the application to practical problems of the scientific studies which it has supported.

Permafrost

Permanently frozen ground can only properly be studied in the field. Accordingly, the start of work in permafrost research has been somewhat slow but with the Committee's support, the Division of Building Research of the National Research Council, jointly with the Canadian Army, initiated field studies of permafrost with special reference to foundations in the North of Canada. Study has been made of the existing records of work already done in this important field and it is hoped to see a steady extension of this important branch of foundation research work. In 1953, a small research station was established at Norman Wells in the North West Territories by the Division of Building Research. It is hoped that a nucleus has been formed for providing a stimulus to the investigation of permafrost.

Snow and Ice

Although snow and ice may appear, at first sight, to be somewhat remote from the specific objectives of the Committee, the association of the term snow mechanics with soil mechanics in the title of the Committee has already elicited appreciative comment from Swiss experts who are acknowledged leaders in the field of snow and ice research. It took the Swiss workers ten years to discover the close relation between soil mechanics and the physical study of snow as a material; they have confirmed the desirability of keeping Canadian work in these two fields closely associated. Representatives of the Committee visited the Swiss Institute for Snow Research and Avalanche Prevention at Davos. Following this visit, the Division of Building Research was pleased to have Dr. M. R. de Quervain, present Director of the Institute, on its staff for one year. As a result of his work in Canada, Dr. de Quervain wrote a report which provides a basis for the development for future snow and ice research work in Canada.

So many interests are involved in the practical problems concerned with snow and ice that the Committee sees ahead of it a real task in co-ordinating research studies in this field, as they may be carried out in the future. As an initial step, the Committee was able to arrange for the taking of regular observations of all the principal characteristics of the snow-cover for four winters (1947-1950). These observations were made by officers of the Meteorological Division, Department of Transport, at twelve selected stations across Canada. This snow survey, a pioneer effort, has demonstrated clear patterns in the characteristics of snow in Canada. The work is continuing at a smaller number of stations and is being extended to other stations in sheltered areas.

Under the auspices of the Committee, a classified system for snow has been established. The Committee has undertaken the task of publishing and distributing this classification system in three languages, English, French, and German. The World Meteorological Organization and the International Union of Geodesy and Geophysics have given their approval and support to this work.

International Relations

In view of the relative newness of the subjects with which it is concerned, the Committee has been particularly anxious to develop fruitful international liaison in order to ensure the best possible correlation of Canadian work with

that in other countries in order to avoid all possible duplication of effort. This has been a most satisfying aspect of the Committee's work. By direction of the Council, the Committee is the National Committee for Canada of the International Society of Soil Mechanics and Foundation Engineering and thus links Canadian work in this important field with that of all other countries of the world. Correspondingly, the Committee has been closely associated with the Commission on Snow and Ice of the International Association of Hydrology which is a constituent part of the International Union of Geodesy and Geophysics. Personal contacts have been established with workers associated with muskeg in Scandinavia and Great Britain. Quite naturally very close associations already exist between the Committee and many American organizations, particularly with reference to permafrost studies in which both countries are so vitally interested.

Operation of Committee

It must finally be made clear that the Committee does not itself undertake any research work. On behalf of the National Research Council it attempts to support existing work, financially when necessary, to promote new work where this is soon to be necessary, and in particular to ensure the best possible correlation between all Canadian workers in the field which it serves. The Committee works closely with the Division of Building Research of the Council, the Soil and Snow Mechanics Sections of which provide secretarial assistance to the Committee. These Sections and the Building Practice Group of the Division are steadily increasing the volume of information on the four main interests of the Committee and this is now available for public use and reference.

The Committee makes its work known by means of a series of technical memoranda, copies of which are available to those interested in the subjects treated. Requests for information on the work of the Committee should be addressed to: The Secretary, Associate Committee on Soils and Snow Mechanics, c/o Division of Building Research, National Research Council, Ottawa, Ontario.

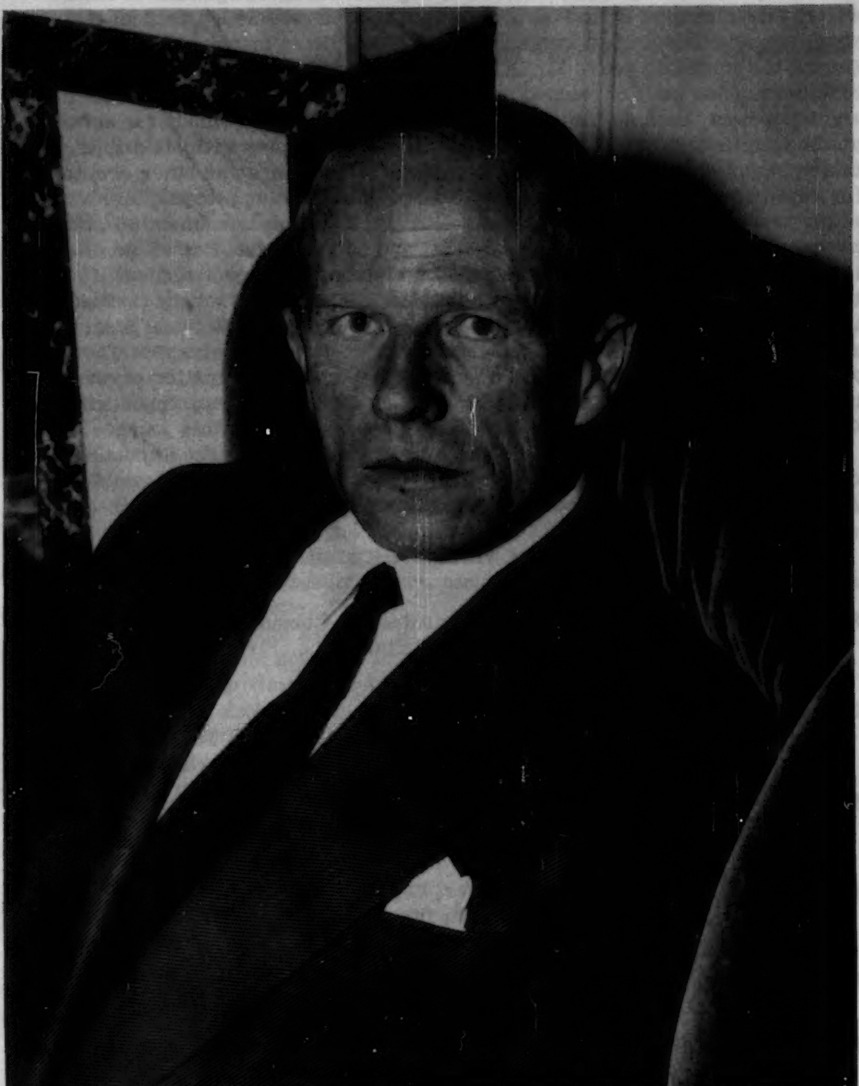
MEMOIR*

WALTER KJELLMAN

Mr. Walter Kjellman, Director of the Swedish Geotechnical Institute, died after a short illness on 17 September 1955. Mr. Kjellman was one of the pioneers and leading personalities in the development of soil mechanics, and his untimely death is a great loss to foundation engineers throughout the world.

Mr. Kjellman was born on 29 November 1905 and obtained his degree in civil engineering at the Royal Institute of Technology, Stockholm, in 1928. From 1930 to 1936 he was employed by the consulting engineering firm "Vattenbyggnadsbyrån" in Stockholm and was primarily engaged in design of the hydro-electric power plant Svir 3 in Russia, except for a short period in

* We should like to express our gratitude to Dr. M. Juul Hvorslev, at the Waterways Experiment Station in Vicksburg, for accepting our request to prepare this memoir of Walter Kjellman.



Walter Kjellman

1930-1931 spent on special investigations in Professor Terzaghi's soil mechanics laboratory in Vienna, Austria. In 1938 he became geotechnical engineer for the Board of Roads and Waterways in Sweden, and he was appointed director of the Swedish Geotechnical Institute when it was established in 1944.

Mr. Kjellman's first contribution to engineering investigations of soils was the development of and tests with a triaxial device with a cubical test specimen and independent control of each of the three principal stresses. During the following years Mr. Kjellman personally or in collaboration with members of his staff developed a great variety of new and ingenious methods and equipment, such as the cardboard-wick and vacuum methods for accelerated consolidation of fine-grained soils, the soil sampler with steel foils for taking samples of very great length, cone penetrometers and other equipment for rapid and continuous sounding of soils, equipment for pile loading tests, shear tests, compression tests, consolidation tests, measurement of earth pressure at rest, pore-water pressures, and settlements. One of his cherished projects was the establishment of an international geotechnical literature classification system and service. Mr. Kjellman had a very fertile and inquiring mind, and he was not afraid of presenting unconventional ideas, as shown in his papers on, "Do Slip Surfaces Exist?" and "Unorthodox Thoughts about Filter Criteria." The last mentioned paper was written for presentation as one of a series of lectures he was scheduled to give in Yugoslavia in October 1955, but he collapsed during the preparations for this journey.

Mr. Kjellman was awarded the Polhem Prize in 1936 by the Swedish Association of Engineers and Architects, First Prize in 1940 by the Royal Swedish Academy of Engineering Sciences, and he was elected member of this Academy in 1954.

The most important papers of which Mr. Kjellman was author or co-author are the following:

Kjellman, W., 1936. Report on an Apparatus for Consummate Investigation of the Mechanical Properties of Soils. Proc. First Int. Conf. Soil Mech., 2:16

Kjellman, W., 1940. Säkerhetsproblemet ur principiell och teoretisk synpunkt (The Problem of Safety from a Fundamental and Theoretical Point of View). Proc. Royal Swed. Inst. Eng. Res., Stockholm, No. 156.

Kjellman, W., 1948. Accelerating Consolidation of Fine-grained Soil by Means of Cardboard Wicks. Proc. Sec. Int. Conf. Soil Mech., 2:302.

Kjellman, W., Kallstenius, T. & Wager, O., 1950. Soil Sampler with Metal Foils. Device for Taking Undisturbed Samples of Very Great Length. Proc. Royal Swed. Geot. Inst., No. 1.

Kjellman, W. & Liljedahl, Y., 1951. Device and Procedure for Loading Tests on Piles. Proc. Royal Swed. Geot. Inst., No. 3.

Kjellman, W., 1952. Consolidation of Clay by Means of Atmospheric Pressure. Proc. Conf. Soil Stabil., Mass. Inst. Tech., Cambridge, Mass., p. 258.

Kjellman, W., Cadling, L. & Flodin, N., 1953. A New Geotechnical Classification System. Proc. Royal Swed. Geot. Inst., No. 6.

Kjellman, W., 1953. Do Slip Surfaces Exist? Proc. Europ. Conf. Stabil. Earth Slopes, Stockholm, 1:14 and Geotechnique (1955) 5 : 1 : 18.

Kjellman, W. & Jakobson, B., 1955. Some Relations between Stress and Strain in Coarse-grained Cohesionless Materials. Proc. Royal Swed. Geot. Inst., No. 9.

Kjellman, W., Kallstenius, T. & Liljedahl, Y., 1955. Accurate Measurement of Settlements. Proc. Royal Swed. Geot. Inst., No. 10.

Kjellman, W., 1955. Unorthodox Thoughts about Filter Criteria. Paper prepared for a lecture in Yugoslavia in October 1955.

Visit of Dr. and Mrs. Laurits Bjerrum to North America

The April Newsletter carried what had to be limited to a preview of the North American tour of points of geotechnical interest by Dr. Laurits Bjerrum, of Norway.

The greater part of the trip has now been completed, and we can report that the itinerary has been slightly more impressive even than the staggering one originally anticipated. And as a result of the kind cooperation of Dr. Leonardo Zeevaert of Mexico City, Mr. Carl B. Crawford of Ottawa, Canada, and Professor Tscheboratioff and Messrs. W. J. Turnbull and W. G. Holtz at Princeton, Vicksburg, and Denver, we are in a position to offer a few interesting details:

After an abbreviated stay in New York and Washington (to which they will return before their reembarkation for Norway) and a week-end visit at the home of Professor Tschebotarioff in Princeton, Dr. and Mrs. Bjerrum spent April 10 to 16 at the Waterways Experiment Station in Vicksburg. Here, among other activities, Dr. Bjerrum visited a number of river control projects, observed some pile loading tests, and inspected construction at Ferrells Bridge Dam, near Jefferson, Texas.

On to Mexico City from April 18 to 24, where, in addition to the many foundation problems and points of natural interest in soil mechanics in the surrounding area, the visitors had time to look at the floating gardens of Xochimilco and the ancient Toltec city of Teotihuacan.

The official record is resumed in Denver on April 28 (On the map it doesn't look like a four-day trip from Mexico City to Denver; but according to Dr. Bjerrum, this isn't bad time at all—especially if you do a little sight-seeing around Los Angeles on the way). Dr. Bjerrum visited the Bureau of Reclamation in Denver and the University of Colorado in Boulder; and on May 2 spoke on the subject of Norwegian landslides at a meeting at the University under the joint sponsorship of the Convocations Committee of the University and the Colorado Section of the Soil Mechanics and Foundations Division, ASCE.

From Denver the Bjerrums made their next stop in Wilmette, Illinois, at the home of Jorj Osterberg, and spent several days in the Chicago area. During this time Dr. Bjerrum gave two talks at Northwestern University, one on Norwegian landslides and one on Norwegian experiences in the investigation of shear strength and slope stability in normally consolidated and slightly over-consolidated clays.

The University of Illinois and Purdue University were the next two destinations, after which the obviously indefatigable travellers headed for Canada for five days, May 14 to 18, in Ottawa, Montreal, and environs. One talk in each of these cities was scheduled, as well as several informal discussions and a number of visits to nearby points of interest, including St. Lawrence Seaway developments, landslide areas near Montreal, and Niagara Falls.

According to a telephone report from Harl Aldrich, our spy in the far east, the Bjerrums are in Cambridge as this is written, apparently still going

strong. Details of the final two weeks of their tour, scheduled for the Boston and New York areas, will be included in the next Newsletter.

Cocktail parties, dinners, and a continuing series of lively and interesting informal discussions have been the order of the day everywhere; and the exchange of information on Norwegian experiences and their counterparts on this continent has been very rewarding to all who have had the chance to participate.

For the record, two points that were not made clear in the April article should perhaps be noted: First, Dr. Bjerrum's title is Director of the Norwegian Geotechnical Institute in Oslo; and second, both Dr. and Mrs. Bjerrum are natives of Denmark. It appears, from evidence offered as well as exemplified by Dr. Bjerrum, that a Norwegian mantle fits a Danish form as easily as the reverse.

The opportunity to meet both Dr. and Mrs. Bjerrum has been a source of genuine enjoyment socially as well as professionally, to all with whom they have come in contact; and their many friends in North America will look forward with pleasure to future meetings.

Finnish Geotechnical Expert Visiting United States and Canada

Professor K. V. Helenelund, Professor at the Finland Institute of Technology in Helsinki, and Chief of the Geotechnical Laboratory of the State Institute for Geotechnical Research, arrived in New York on May 15 to begin a tour of points of interest to him in Canada and the United States that is scheduled to occupy him until July 6. Professor Helenelund is one of Finland's most prominent experts in soil mechanics and related subjects.

He has already visited Harvard University and Massachusetts Institute of Technology; and his plans include visits to New York, Princeton, Vicksburg, Denver, Chicago, Urbana, Ottawa, and Montreal.

Local Section Notes

On April 13, 1956, a Committee on Soil Mechanics and Foundations, of the Pittsburgh Section, ASCE, was organized in accordance with Society policy. Officers of the Pittsburgh Section Committee on Soil Mechanics and Foundations are: Chairman, Elio D'Appolonia, of the Carnegie Institute of Technology; Vice Chairman, Alfred C. Ackenheil, of the University of Pittsburgh; and Secretary, Gerald A. Oakes, of Richardson, Gordon and Associates. Regular luncheon meetings are scheduled on the second Tuesday of each month. Meanwhile, the Committee is making arrangements for an educational program on soil mechanics and foundations to be held at Carnegie Institute of Technology and the University of Pittsburgh in the spring of 1957. Six two-hour sessions on successive Tuesdays and Thursdays of a three-week period are planned to cover various aspects of geology, soil exploration, soil testing, and foundations. Speakers will be selected from the Pittsburgh Section, ASCE; and in addition to ASCE members, members of the Pennsylvania Society of Professional Engineers, AIA, and AGCA will be invited to attend.

Soils Conferences in South Africa and New Zealand

The Newsletters of March and September, 1955, contained announcements of soils conferences scheduled for some rather remote parts of the world—remote, that is, from the provincial point of view of one sitting on the shore of Lake Michigan. But soil mechanics allots problems impartially to all continents, so perhaps our readers will be interested in a follow-up of those announcements.

The First Southern African Regional Conference on Soil Mechanics and Foundation Engineering was held in Pretoria, South Africa, October 11-14, 1955, under the Chairmanship of Mr. J. E. Jennings. The papers presented there were divided into three sections:

I - Foundations for Buildings on Expansive Soils; II - Foundations for Roads and Airport Runways; and III - Soil Mechanics Problems of Special Significance in South Africa. Mr. Jennings' detailed report of the Conference appears in Géotechnique for March, 1956; and the papers have been published in the September and December, 1955, issues of the Transactions of the South African Institution of Civil Engineers. Inquiries may be directed to the Division of Soil Mechanics and Foundation Engineering of that Institution, at P. O. Box 1291, Pretoria, South Africa.

The Second Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering was held at Canterbury University College, Christchurch, New Zealand, January 23-26, 1956, under the joint sponsorship of the University of New Zealand and the New Zealand Institution of Engineers. Reported attendance numbered about seventy from New Zealand, twenty from Australia, and one from Canada. Twenty-five papers were presented at the Conference, covering a variety of subjects, especially earth dams, which have become a matter of considerable interest in Australia and New Zealand. Seventeen of these papers have been published in the November, 1955, issue of New Zealand Engineering (the Journal of the New Zealand Institution of Engineers).

Fourth International Conference on Soil Mechanics and Foundation Engineering

The International Society of Soil Mechanics and Foundation Engineering has recently published Bulletin No. 1, dated January, 1956, on the Fourth International Conference scheduled for August 12-24, 1957, in London. Copies may be obtained through John Lowe III, Secretary, U. S. National Council on Soil Mechanics and Foundation Engineering, Room #400, 62 West 47th Street, New York 36, New York, or directly from Mr. A. Banister, Secretary, International Society of Soil Mechanics and Foundation Engineering, Institution of Civil Engineers, Great George Street, London, S.W. 1, England.

Scheduled ASCE Conventions

Pittsburgh Convention

William Penn Hotel October 15-19, 1956.

Jackson Convention

Hotel Heidelberg
Jackson, Mississippi

February 18-22, 1957.

October Newsletter

Deadline date for arrival at this office of contributions for the October Newsletter: August 20, please.

Howard P. Hall, Editor
Department of Civil Engineering
Northwestern University
Evanston, Illinois.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

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JULY: 732(ST), 733(ST), 734(ST), 735(ST), 736(ST), 737(PO), 738(PO), 739(PO), 740(PO), 741(PO), 742(PO), 743(HY), 744(HY), 745(HY), 746(HY), 747(HY), 748(HY)[c], 749(SA), 750(SA), 751(SA), 752(SA)[c], 753(SM), 754(SM), 755(SM), 756(SM), 757(SM), 758(CO)[c], 759(SM)[c], 760(WW)[c].

AUGUST: 761(BD), 762(ST), 763(ST), 764(ST), 765(ST)[c], 766(CP), 767(CP), 768(CP), 769(CP), 770(CP), 771(EM), 772(EM), 773(SA), 774(EM), 775(EM), 776(EM)[c], 777(AT), 778(AT), 779(SA), 780(SA), 781(SA), 782(SA)[c], 783(HW), 784(HW), 785(CP), 786(ST).

SEPTEMBER: 787(PO), 788(IR), 789(HY), 790(HY), 791(HY), 792(HY), 793(HY), 794(HY)[c], 795(EM), 796(EM), 797(EM), 798(EM), 799(EM)[c], 800(WW), 801(WW), 802(WW), 803(WW), 804(WW), 805(WW), 806(HY), 807(PO)[c], 808(IR)[c].

OCTOBER: 809(ST), 810(HW)[c], 811(ST), 812(ST)[c], 813(ST)[c], 814(EM), 815(EM), 816(EM), 817(EM), 818(EM), 819(EM)[c], 820(SA), 821(SA), 822(SA)[c], 823(HW), 824(HW).

NOVEMBER: 825(ST), 826(HY), 827(ST), 828(ST), 829(ST), 830(ST), 831(ST)[c], 832(CP), 833(CP), 834(CP), 835(CP)[c], 836(HY), 837(HY), 838(HY), 839(HY), 840(HY), 841(HY)[c].

DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(SU), 858(BD), 859(BD), 860(BD).

VOLUME 82 (1956)

JANUARY: 861(SM1), 862(SM1), 863(EM1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1), 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)[c], 877(HW1)[c], 878(ST1)[c].

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1), 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1), 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904(PO1)[c], 905(SA1)[c].

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913(WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)[c].

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941(SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958(SA2), 959(PO2), 960(PO2).

MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW3), 968(WW3), 969(WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2).

JUNE: 990(PO3), 991(PO3), 992(PO3), 993(PO3), 994(PO3), 995(PO3), 996(PO3), 997(PO3), 998(SA3), 999(SA3), 1000(SA3), 1001(SA3), 1002(SA3), 1003(SA3)[c], 1004(HY3), 1005(HY3), 1006(HY3), 1007(HY3), 1008(HY3), 1009(HY3), 1010(HY3)[c], 1011(PO3)[c], 1012(SA3), 1013(SA3), 1014(SA3), 1015(HY3), 1016(SA3), 1017(PO3), 1018(PO3).

JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023(ST4), 1024(ST4)[c], 1025(SM3), 1026(SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

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c. Discussion of several papers, grouped by Divisions.